PREDICTION OF ULTIMATE SIDE SHEAR FOR DRILLED SHAFTS IN MISSOURI SHALES

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**PREDICTION OF ULTIMATE SIDE SHEAR FOR DRILLED SHAFTS IN
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ABSTRACT

Bridges crossing major rivers in the State of Missouri have relied heavily on
drilled shafts socketed into bedrock as the principal means of achieving bearing capacity
of the foundation elements. Rock sockets in shales and weak rocks are designed to
develop axial capacity in side shear. End bearing is usually neglected. The current
procedures used by the Missouri Department of Transportation (MoDOT) to estimate the
ultimate unit side shear follow procedures in the 1996 AASHTO and FHWA-IF-99-025
manuals. The methods provided in these manuals roughly predict the ultimate unit side
shear to be equal to 0.15 times the unconfined compressive strength (q_u) of the shale
core. These design methods have lead to the design of rock sockets 1.5 to 2.5 meters (5
to 8 ft) in diameter and as long as 15 meters (50 ft) to support loads in the range of 112 to
169 MN (1000 to 1500 tons). In order to achieve more economical designs and to take
some of the uncertainty out of the prediction of the ultimate unit side shear, MoDOT
conducted four full-scale Osterberg cell (O-cell) load tests at three different sites.

Analysis of the load test data indicated that the ultimate unit side shear may be
conservatively estimated as 0.3 times the unconfined compressive strength of the shale.
Design methods proposed by Rowe and Armitage (1987) and Kulhawy and Phoon (1993)
produced almost identical results and most closely predicted the ultimate unit side shear
measured in the load tests. Because the Rowe and Armitage (1987) method slightly over-

estimates the ultimate unit side shear for shale with low compressive strengths, a minor modification of the method is proposed to produce slightly more conservative values. Finally, the analysis of the load test data will lead to significant increases in the predicted ultimate unit side shear over current methods followed by MoDOT.

CHAPTER ONE
INTRODUCTION

1.1 Problem Statement

Design of foundations for bridges in the State of Missouri have relied heavily on drilled shafts socketed into bedrock as the principal means of achieving bearing capacity of the foundation elements. Rock sockets in shales and weak rocks are designed to develop axial capacity in side shear. End bearing is usually neglected. Current procedures used by the Missouri Department of Transportation (MoDOT) to estimate the ultimate unit side shear capacity in weak rock often lead to exceedingly long rock sockets. As a result, MoDOT has conducted four Osterberg Load cell tests on large drilled shafts at three different bridge sites. These load tests and the results they have produced are the subject of this thesis. The objective of this thesis is to document the load tests and to present an evaluation of several design methods based on the results of these tests.

1.2 Organization of Thesis

Existing empirical and analytical methods for predicting the ultimate unit side shear capacity for drilled shafts socketed into weak rock are presented in Chapter 2. Empirical methods are generally based on results of full-scale load tests while analytical methods attempt to model the soft rock-drilled shaft interface numerically, often using the finite-element method.

All load tests presented in this thesis were performed using the Osterberg cell (O-cell^{TM}) method of loading, invented by Dr. Jorj Osterberg. The general procedures used to perform and interpret O-cell load tests are described in Chapter 3, along with

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associated instrumentation such as strain gages and telltales used to determine end bearing and side shear capacities of drilled shafts.

The results of the four load tests are presented in Chapters 4, 5, and 6. In each of these chapters, the general geology of the site area is first described followed by more thorough descriptions of the specific strata involved in the load tests. The procedures followed to construct the test shafts and perform the load tests are then described along with presentation of the shaft layout and associated instrumentation. Finally, the results of each load test are presented.

In Chapter 4, load tests performed on two full-scale drilled shafts for a proposed bridge across the Missouri River at Lexington, Missouri are described. The bedrock at the Lexington site consists of Pennsylvanian Age shales, siltstones, sandstones, limestones, and scattered coal beds. The shafts were tested to maximum loads of 13.3 MN (1,495 tons) and 17.5 MN (1,968 tons) in May and June of 1999. The Osterberg cell load tests where successful in allowing MoDOT to develop a more economical design for the drilled shafts for the proposed bridge.

Chapter 5 presents the results of a load test performed on a full-scale drilled shaft as part of the reconstruction of an interchange in the Kansas City metropolitan area known as the Grandview Triangle. The bedrock at the Grandview Triangle site consists of horizontally bedded layers of limestones and shales known as the Kansas City Group. The shaft was loaded to 34.3 MN (3,856 tons) on June 3, 2002. Data from the Osterberg cell load test would allow the 2.3 m (7.5 ft) diameter rock sockets at bridge A6252 to be shortened a total of 65.2 m (214 ft) for a net savings of $19,000.

A load test on a "production" drilled shaft for a proposed bridge across the Missouri River at Waverly, Missouri is described in Chapter 6. The bedrock at the Waverly site consists of Pennsylvanian Age shales, siltstones, sandstones, limestones, and scattered coal beds. The production shaft was tested to a maximum load of 22.5 MN (2,525 tons), on September 30, 2002. The Osterberg cell load test was successful in testing the shaft to twice the design load and assuring the foundation engineers that the main river pier would be safe.

In Chapter 7, the results of all four tests are summarized with particular focus on values of unit side shear determined from the load tests for various strata. The measured unit side shear values are then compared to values predicted by several current design methods and a method to more accurately predict ultimate unit side shear in Missouri shales is proposed. Finally, Chapter 8 includes a summary of this thesis, conclusions reached from the four load tests described, and several recommendations for further work.

CHAPTER TWO
METHODS FOR PREDICTING SIDE SHEAR IN ROCK

Empirical and analytical methods for predicting the unit side shear capacity of drilled shafts socketed into weak rock are presented in this chapter. Empirical methods are generally based on results from full-scale load tests while analytical methods attempt to model the soft rock-drilled shaft interface numerically, often using finite-element solutions.

Rock socketed drilled shafts transfer axial load through upper non-competent strata to competent bedrock, which can sustain the load. The load is transferred to the bedrock through two basic load bearing mechanisms, end bearing and side shear (Kiehne 1997). Drilled shafts designed to carry load in end bearing require construction and inspection techniques that guarantee the cleanliness of the base (Pells 1980). This may be difficult to achieve, particularly for deep sockets that use a drilling fluid such as water or slurry. Rock socketed end bearing drilled shafts normally require competent rock which can support large loads for at least two shaft diameters below the base of the rock socket. For weak rocks such as shales that cannot carry large loads in end bearing, rock socketed drilled shafts are designed to carry axial load primarily in side shear. The ultimate unit side shear may be related to factors created by construction technique such as interface roughness and cleanliness, properties of the weak rock, such as cohesion and angle of internal friction, and the geometry of the rock socket.

2.1 Side Shear

Many designers prefer to design drilled shafts to take load in side shear only versus combined side shear and end bearing because the amount of movement required to

mobilize side shear is relatively small, while that required to mobilize end bearing is relatively large (Osterberg 2000). Side shear is generally fully mobilized when shaft movement is 6 to 13 mm (1/4 to 1/2 inch) while end bearing is not fully mobilized until the movement is on the order of centimeters (inches).

Both empirical and analytical methods have been used to predict the ultimate unit side shear of rock sockets (Carruba 1997). Empirical methods are generally based on full-scale load tests in which the ultimate unit side shear is back-calculated from instrumentation. The ultimate unit side shear (f_{max}) is then related to the unconfined compressive strength of the soil/rock (q_u) using an empirical constant, usually denoted α, as

$$f_{max} = \alpha \cdot q_u \qquad\qquad (2.1)$$

Other researchers have attempted to address drilling parameters such as rock socket roughness by adding a second constant, β. In this case the empirical relation takes the form

$$f_{max} = \alpha \cdot \beta \cdot q_u \qquad\qquad (2.2)$$

Still other researchers believe the true expression relating unit side shear to unconfined compressive strength is a power function of the form

$$f_{max} = \alpha \cdot (q_u)^c \qquad\qquad (2.3)$$

Analytical methods are often based on finite element methods and are generally similar in form to equations 2.1 through 2.3 with additional factors to address roughness, initial normal stress at the shaft rock interface, stiffness and cohesion of the rock mass,

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and the presence of joints or seams in the rock mass. Specific empirical methods are described in Section 2.2; analytical methods are described in Section 2.4.

2.2 Empirical Methods

The following empirical methods have been developed based on data from the geographic area and/or rock formations of interest to the authors. As with all empirical methods, additional calibration should be performed using full-scale load tests for geographic areas or rock formations that are significantly different than those used to develop the methods.

2.2.1 Rosenberg and Journeaux

The Rosenberg and Journeaux (1979) method is based on a top-down load test performed on a 457 mm (18 in) diameter rock socket in highly fractured Andesite with an unconfined compressive strength of 10.3 MPa (108 tsf) and a pullout test on a 203 mm (8 in) diameter rock socket in shale with an unconfined compressive strength of 20.7 MPa (216 tsf). A 0.1 m (4 in) styrofoam isolating pad was placed at the base of the 560 mm (22 in) long rock socket for the top-down load test to eliminate end bearing. Rosenberg and Journeaux correlated their test data with tests performed by Moore (1964), Matich and Kozicki (1967), Thorburn (1966), Seychuck (1970), Gibson and Deveny (1973), and Jackson et al. (1974) in shales and sandstones in Canada. They found that the ultimate unit side shear was best predicted as

$$f_{max} = 1.11 \; (\; q_u \;)^{0.51} \hspace{3cm} (2.4)$$

where f_{max} and q_u are given in tsf.

2.2.2 Horvath and Kenney

Horvath and Kenney (1979) developed a method based on data from 50 sites in Australia, Canada, England, and the U.S. They found that ultimate unit side shear was best predicted as

$$f_{max} = \alpha \, (\, q_u \,)^{0.5} \qquad\qquad\qquad (2.5)$$

where α is 2.5 to 3 for shafts greater than 16 inches in diameter and f_{max} and q_u are given in psi. For SI units, α is 0.2 to 0.25 for shafts greater than 406 mm (16 in) in diameter and f_{max} and q_u are given in MPa.

Horvath et al. (1983) subsequently proposed a modification to address borehole roughness for artificially roughened boreholes by evaluating α based on the depth of the grooves in the rock socket as shown in Figure 2.1. They found that the coefficient α could be computed as

$$\alpha = RF^{\,0.45} = 0.8 \, [\, \Delta r/r \, (\, L_t \,/\, L_s \,)]^{\,0.45} \qquad\qquad (2.6)$$

where RF is a dimensionless roughness factor, Δr is the average height of the asperities or grooves, r is the nominal socket radius to the base of the grooves, L_t is total distance along the socket wall profile, and L_s is the nominal socket length.

Socket roughness may be determined approximately in the field with either mechanical or electronic calipers. Sonic calipers may be used in dry holes while sonar calipers are available for drilled shafts constructed using either water or drilling slurry. Recently, a laser bore-hole caliper has been developed in Australia (Seidel 1998). Reese and O'Neill (1988) define a socket as rough if the roughness factor, RF, exceeds 0.10.

L_s = Nominal socket length

r

L_t = Total distance along rough interface

Δr
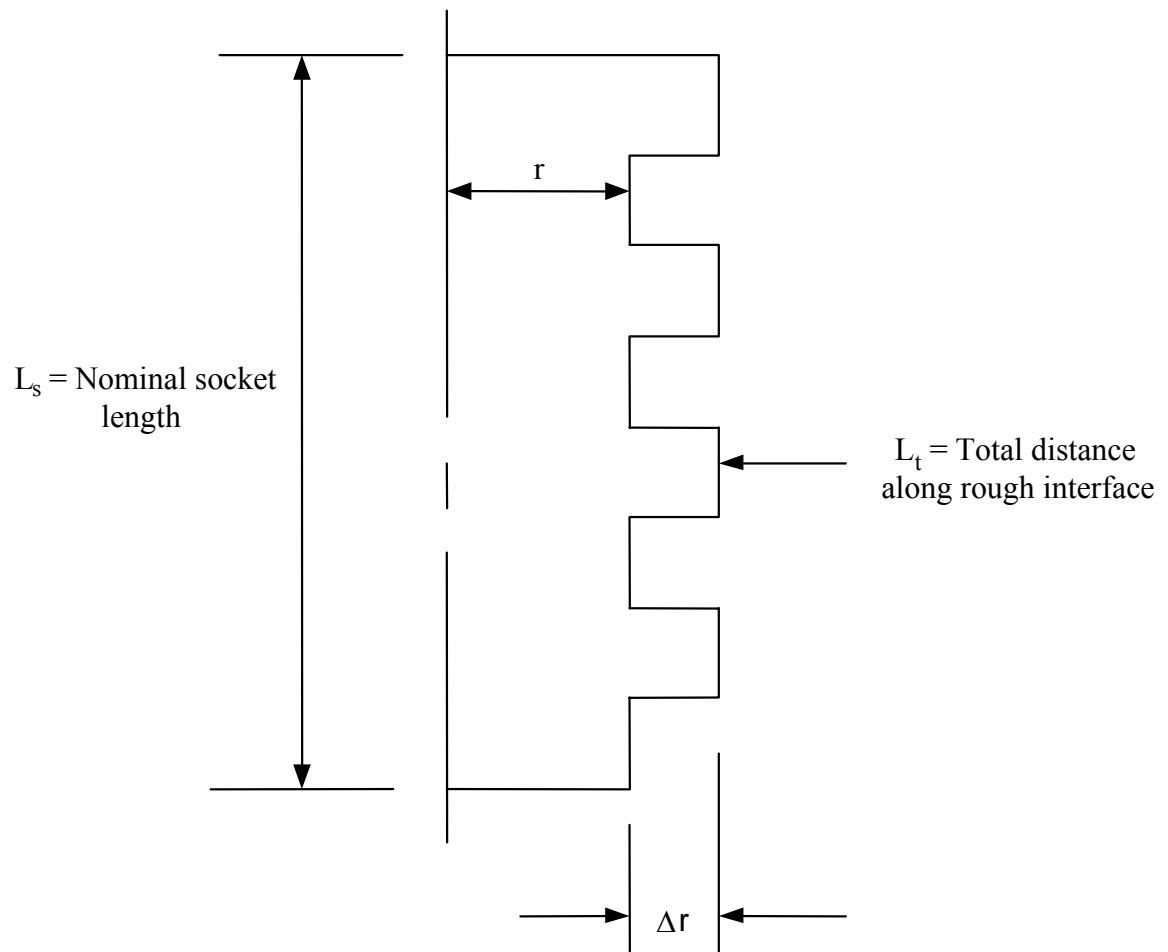
Figure 2.1- Schematic of rock socket parameters (after Horvath et al. 1983).

2.2.3 Williams et al. and Williams and Pells

Williams and his colleagues (Williams et al. 1980; Williams and Pells 1981) developed a semi-empirical method based on 15 load tests on shafts with diameters ranging from 335 to 1580 mm (13 to 62 in) founded in the Silurian-aged Mudstone in Melbourne, Australia and 27 load tests on drilled shafts with diameters ranging from 64 to 710 mm (2.5 to 28 in) in the Hawkesbury sandstone in Sydney. They found that f_{max} could be estimated as

$$f_{max} = \alpha \cdot \beta \cdot q_u \qquad (2.7)$$

where α reflects variations in the intact strength of the rock only and can be obtained from Figure 2.2, and β is an adjustment factor to account for seams of softer material in the rock. If laminations are closed tightly, β is taken to be 1.0. Otherwise β is obtained from Figure 2.3, where

$$E_m / E_c = \frac{L_c}{(E_c / E_s)\, \Sigma t_s + \Sigma t_c} \qquad (2.8)$$

where E_m is Young's modulus of the rock mass, E_c is Young's modulus of intact rock cores, and E_s is the Estimated Young's modulus of the material in seams (all given in MPa) L_c is the length of core, t_s is the thickness of each seam, and t_c is the thickness of intact rock (all given in either mm or meters).
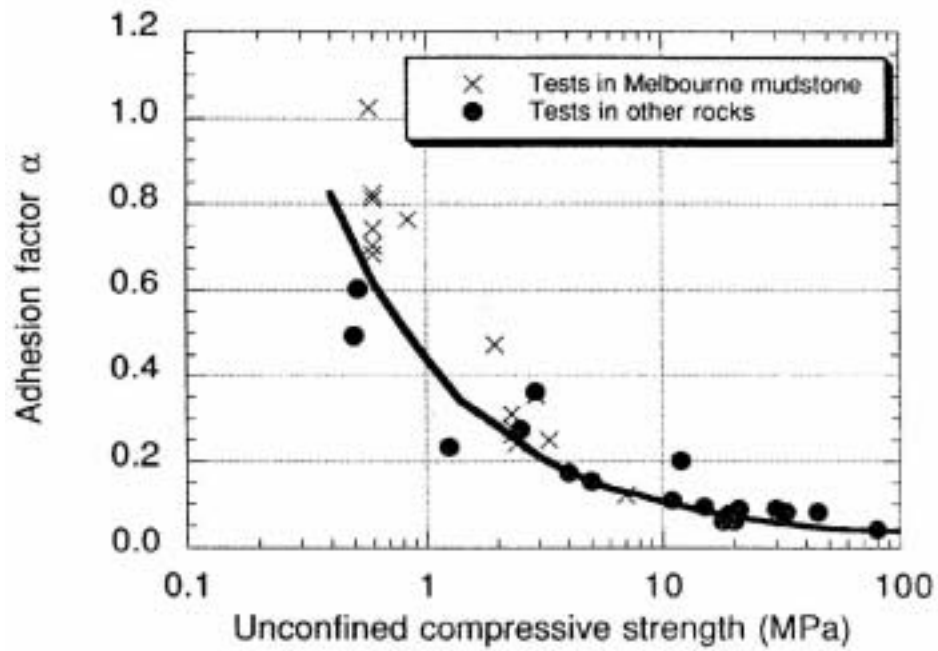
Figure 2.2- Adhesion factor α versus unconfined compressive strength (from Williams et al. 1980).

Mass Modulus factor $= E_m / E_c$
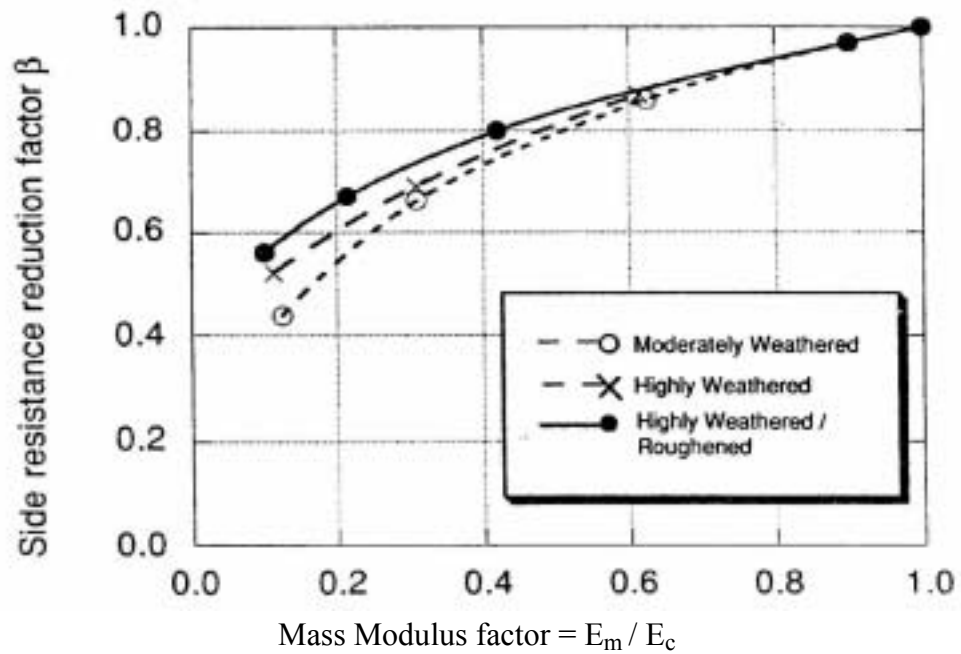
Figure 2.3- Side resistance reduction factor β versus mass modulus factor (from Williams et al. 1980).

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2.2.4 Rowe and Armitage

Rowe and Armitage (1987) proposed a method that assumes the interface is clean and the side of the shaft is in either a natural state or artificially roughened. This method was confirmed based on load tests on drilled shafts with relatively smooth interfaces in Ordovician aged shales in southern Ontario Canada. The equations they proposed are based on a shaft roughness classification by Pells et al.(1980) as defined in Table 2.1. For roughness classes R1, R2, and R3, they propose computing f_{max} as

$$f_{max} = 0.45 \, (\, q_u \,)^{0.5} \qquad\qquad (\,2.9\,)$$

where f_{max} and q_u are in MPa. For roughness class R4, the equation is

$$f_{max} = 0.6 \, (\, q_u \,)^{0.5} \qquad\qquad (\,2.10\,)$$

Table 2.1= Roughness Classification (after Pells et al. 1980).

Roughness Class	Description
R1	Straight, smooth sided socket, grooves or indentions less than 1 mm deep.
R2	Grooves of depth 1 to 4 mm, width greater than 2 mm, at spacing 50 – 200 mm.
R3	Grooves of depth 4 to 10 mm, width greater than 5 mm, at spacing 50 – 200 mm.
R4	Grooves or undulations of depth greater than 10 mm, width greater than 10 mm, at spacing 50 – 200 mm.

2.2.5 Reese and O'Neill

Reese and O'Neill (1988) proposed a method for the Federal Highway Administration (FHWA) that is derived from a method developed by Kulhawy (1983) in an earlier FHWA publication on drilled shafts. The 1988 FHWA method conservatively recommends assuming that the load is carried entirely in side shear or entirely in end bearing, depending on whether or not the computed settlement is more or less than 10

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mm (0.4 in). Based on load tests in three clay-shale formations, Reese and O'Neill

recommended using Equation 2.5, developed by Horvath and Kenney (1979), for rock

with an unconfined compressive strength greater than 2.01 MPa (21 tsf). For rock with

an unconfined compressive strength of 1.72 MPa to 2.01 MPa (18 to 21 tsf) they

recommended that an equation developed by Carter and Kulhawy (1987) be used to

determine the ultimate unit side shear. The Carter and Kulhawy relationship is given as

$$f_{max} = 0.15 \; q_u \qquad\qquad\qquad (\,2.11\,)$$

where q_u is in the range of $2.01 \geq q_u \geq 1.72$ MPa $(21 \geq q_u \geq 18$ tsf$)$. For rock with an

unconfined compressive strength less than 0.38 MPa (4 tsf) they recommended that

ultimate unit side shear be calculated as

$$f_{max} = 0.275 \; q_u \qquad\qquad\qquad (\,2.12\,)$$

where $q_u \leq 0.38$ MPa (4 tsf). Values of the ultimate unit side shear for q_u between 0.38

MPa and 1.72 MPa (4 tsf and 18 tsf) may be determined by interpolation between

equations 2.11 and 2.12.

2.2.6 Kulhawy and Phoon

Kulhawy and Phoon (1993) used the database developed by Rowe and Armitage

(1984), which consisted of 67 load tests at 18 different sites, supplemented by 47 load

tests from 23 sites in Florida Limestone (McVay 1992) to develop an expression for f_{max}

involving socket roughness. They plotted unit side resistance (f_s) versus average soil and

rock strength normalized by atmospheric pressure (P_a) on a log-log plot for smooth

sockets in soil and for rough sockets in rock as defined by Rowe and Armitage (1987) in

Table 2.1. The results where interpreted as linear, giving the exponential relationship

$$f_{max} = P_a \, \psi \, (q_u / 2P_a)^{0.5} \hspace{4cm} (2.13)$$

where ψ is a dimensionless factor that reflects variations in the intact strength of the

rock and roughness of rock socket. They found the mean value of ψ in rock may be

taken as equal to 2, with a standard deviation of 0.17. They further found that an extreme

lower bound for rock would be 0.5, while 1.0 is a better working lower bound. The

apparent upper bound for ψ is 3, which could be used for very rough or artificially

roughened drilled shafts in rock but should not be used without load tests. Rowe and

Armitage (1984) suggest a mean ψ value of 2 and a value of 2.7 for roughened shafts.

2.2.7 Discussion of Empirical Methods

According to Zhang (1998), relationships for relating the ultimate side shear to

the unconfined compressive strength of the rock follow two major groups. The first is a

simple linear expression of the form

$$f_{max} = \alpha \cdot q_u$$

While the other is a power function of the form

$$f_{max} = \alpha \cdot (q_u)^c$$

Whether the relation between f_{max} and q_u is better represented by a power function or a

linear function depends mainly on the range of q_u considered (O'Neill et al. 1996). The

linear function proposed by Carter and Kulhawy (1987) is only applicable for q_u between

1.7 and 2 MPa (18 to 21 tsf), whereas the power function of Horvath and Kenny (1979)

applies over a wider range. Zhang (1998) concluded that "Extensive studies of load test

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data by Williams and Pells (1981) and Kulhawy and Phoon (1993) indicated that the

power-curve relationship is closer to the real case."

O'Neill et al. (1996) analyzed a database of 139 loading tests and came to the

conclusion that a unique value of α does not exist and more parameters than just q_u are

required to make accurate predictions of f_{max}. Williams and his colleagues, (Williams et

al. 1980; Williams and Pells 1981) developed a semi-empirical method where α reflects

variations in the intact strength of the rock and β is an adjustment factor to account for

seams of softer material in the rock (Eq. 2.7). Due to the difficulty in isolating these

different variables with empirical data, much of the work to evaluate additional factors

has utilized numerical methods as described in the following sections.

2.3 Factors Affecting Unit Side Shear for Drilled shafts in Rock

O'Neill (1996) has noted the ultimate side shear capacity of drilled shafts

socketed into rock is dependent on a number of factors including factors related to

construction technique such as

- interface roughness

- cleanliness of interface

 -initial lateral concrete pressure

- length of time borehole remains open prior to concreting

- destroyed or intact base resistance

factors related to the properties of the rock such as

- angle of internal friction of the rock

- angle of interface dilation

- formation stiffness

- initial coefficient of lateral earth pressure

factors related to the load test method such as

- pull out test

- top-down loading

- jacking upward from the base (O-cell^{TM})

and factors related to socket geometry such as

- length

- diameter

The following sections summarize the current understanding of the effects of these parameters.

2.3.1 Interface roughness

Research conducted by Williams and Pells (1981) and Horvath et al. (1983) indicated that the ultimate unit side shear resistance in drilled shaft sockets in soft cohesive rock is controlled by the interface roughness as much as, or more than rock strength. In general, shafts with rougher side-walls are expected to have higher unit side shear than shafts with smooth side walls. Kulhawy and Phoon (1993) found that the ratio of maximum unit side shear to unconfined compressive strength (f_{max} / q_u) could be four times higher for a rough socket than for a smooth socket. Williams and Pells (1981) and Kodikara (1992) have modeled the borehole roughness by assuming clean triangular interface joints as shown in Figure 2.4. Williams and Pells used a finite element analysis to corroborate their field load test. Kodikara uses a rational mathematical model to account for borehole roughness, strength of the rock, and dilatancy at the shaft-rock

interface. The height of the asperities or grooves (hm) and the angle of the asperities

with the vertical side of the borehole (im) are needed for the model.

In clay shale, the interface is generally not clean due to disturbance by the auger,

perched groundwater or seeps, or even water introduced into the hole by drillers to

facilitate the removal of the cuttings which can produce a smear zone several millimeters

thick (O'Neill and Hassan 1993). Research by O'Neill and Hassan (1993) in the Eagle

Ford Shale in Texas indicates that the borehole sidewalls can be modeled as a sinusoidal

interface pattern as shown in Figure 2.5, particularly for clay shale.
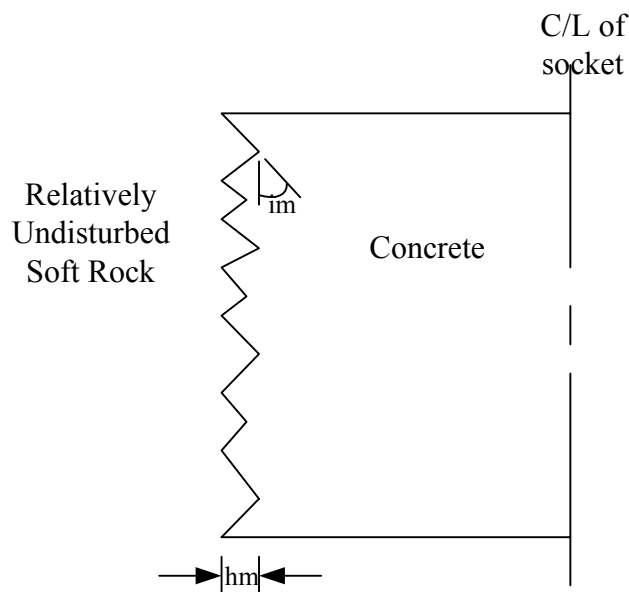
Figure 2.4-   Model of socket roughness for Melbourne Mudstone, where im = mean
value of angle between face of the asperity or groove and the vertical and
hm = mean double-amplitude height of the asperities (after Kodikara et al.
1992).

C/L of
Socket

Rock

Concrete

Smear Zone
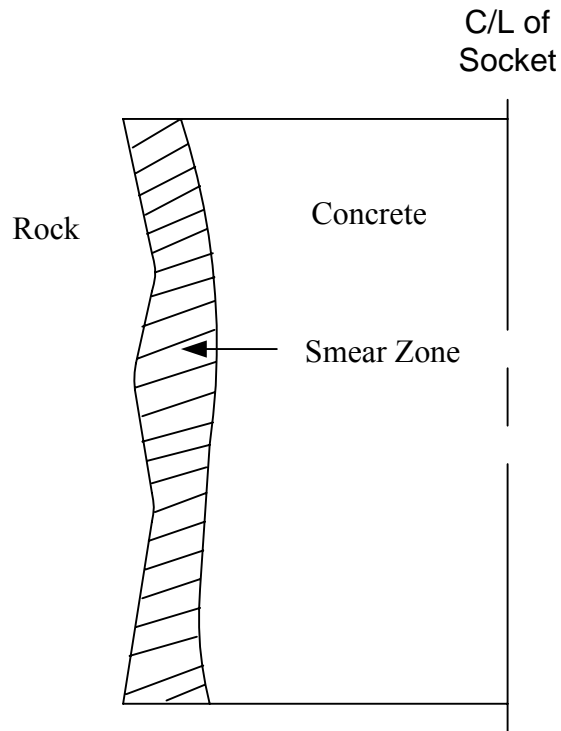
Figure 2.5- Sinusoidal interface pattern in clay shale (after O'Neill and Hassan 1993).

2.3.2 Cleanliness of interface

Hassan and O'Neill (1997) have found that smearing of argillaceous intermediate

geomaterials (IGMs) caused by construction technique has a significant negative effect

on load transfer. The smearing of the interface is believed to account for large

differences observed in f_{max} in full-scale loading tests in IGMs of similar strength. IGMs

or intermediate geomaterials are defined by O'Neill (1996) as argillaceous geomaterials

including heavily overconsolidated clays, clay shales, sapprolites, and mudstones that are

prone to smearing when drilled and have an unconfined compressive strength (q_u) in the

range of 0.5 to 5.0 MPa (5 to 50 tsf). Osterberg (1992) found that a thin grease like

layer is formed, for drilled shafts constructed in shale using water as the drilling fluid,

which can greatly reduce the shaft friction. Hassan and O'Neill (1997) have

recommended that if smearing can occur during the drilling process the shaft be designed as smooth.

Horvath (1980) found that the loss of unit side shear due to smear is not as great in rough sockets, particularly in artificially roughened sockets. Osterberg (1992) reported that drilling methods used in weaker rocks, and even hard sandstones, provide sufficient roughness and there seems to be no effect of smoothness.   Shear rings or grooving of the socket has been recommended by Hummert (1988) to increase unit side shear in the Pierre Shale in Colorado.

2.3.3 Initial lateral concrete pressures

Initial lateral concrete pressures due to the depth of concrete in shaft may have a significant influence on the unit side shear for both rough and smooth sockets. The initial normal stress, σ_n, on the concrete-rock/soil interface at the mid-depth of the socket can be taken as being equal to the pressure of the fluid concrete. Failure at the interface of a smooth socket is primarily a function of the modulus of the rock mass, E_m, the angle of sliding resistance at the interface, ϕ_{rc} , and the initial normal stress at the interface, σ_n. As σ_n increases f_{max} increases significantly (Hassan and O'Neill 1997). A finite element study performed by Hassan and O'Neill (1997) found that the unit side shear in the elastic range increases with increasing values of σ_n, but that there is no major difference in the ultimate unit side shear, f_{max}, when settlement approaches an infinite value. The ultimate unit side shear value tends to be equal to the undrained shear strength (s) of the soft rock or $q_u / 2$ when the angle of internal friction of the soft rock is equal to zero, ($\phi = 0$).

2.3.4 Length of time borehole remains open prior to concreting

For argillaceous geomaterials such as clay shales it is generally believed that the unit side shear is reduced as the time of exposure of the rock socket to drilling fluids is increased. This is primarily caused by softening of the side-walls of the socket. Osterberg (1992) has noted a reduction in unit side shear for shale with rock sockets that were drilled dry and which rapidly deteriorate when exposed to air.

2.3.5 Angle of internal friction and angle of interface dilation

In rough sockets, failure often takes places at the base of the asperities or grooves, by shearing off the asperities, whereas in a smooth socket failure takes place due to slip along the interface between the shaft and rock. As the angle of internal friction (ϕ) of the weak rock increases, the ultimate unit side shear, f_{max}, increases for rough sockets while there is virtually no effect of ϕ for smooth sockets (Hassan and O'Neill 1997). In contrast, since failure occurs at the base of the asperities for rough sockets, the angle of interface dilation (ϕ_{rc}) has little effect on the ultimate unit side shear. For smooth sockets however, where sliding takes place along the interface, the angle of interface dilation can significantly affect the ultimate unit side shear with f_{max} increasing with increasing ϕ_{rc}.

2.3.6 Formation stiffness

The formation stiffness E_m (and by inference q_u) has a significant effect on load transfer in side shear for smooth sockets (Hassan and O'Neill 1997). Goeke (1979) and Osterberg (1999) have concluded that the lab strength of rock cores is lower than the insitu or formation strength. Goeke attributes the erratic lab data partially to swelling of the shale in the core barrel during coring and to partial drying of samples, which result in

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development of micro fissures in the sample. Naturally occurring laminations in shales also cause low unconfined compressive strengths in the lab. Insitu, the weight of the overburden increases the shear strength between the laminations (Osterberg 1999). For weak rocks with naturally occurring laminations, O'Neill (1996) has proposed testing undrained rock cores in compression using a triaxial cell with a confining pressure to more appropriately represent field conditions.

Based on published test data, Osterberg (1992) has found that as the unconfined compressive strength, increases the ultimate unite side shear decreases as a fraction of the rock strength. For weak rocks, such as shales with q_u in the range of 2.4 to 3,447 kPa (3.6 to 36 tsf), the ultimate side shear is 0.3 to 0.5 times q_u. For rocks with strengths in the range of 3.4 to 13.8 MPa (36 to 144 tsf), the ultimate unit side shear is 0.1 to 0.3 times q_u. Finally for rocks such as hard limestone and granite with strengths in the range 13.8 to 55.2 MPa (144 to 576 tsf), the ultimate unit side shear is 0.03 to 0.1 times q_u. This data would again reaffirm that the power-curve relationship for the ultimate unit side shear is closer to the real case.

2.3.7 Socket diameter

The diameter of rock sockets can also affect the unit side shear values. It is generally believed that as the diameter of the socket increases the ultimate unit side shear decreases, but very little comparative load testing has been done on large shafts. A study of existing test data by Horvath and Kenny (1979) indicates the ultimate unit side shear decreases with increasing diameter but for socket diameters larger than 380 mm (15 inches) the effect of socket diameter appears negligible. Expanding cavity theory which

can be used to compute radial stresses caused by dilation of the socket during axial

loading suggests that there is an effect of diameter on load transfer (Hassan and O'Neill

1997). Analysis by Hassan and O'Neill showed that unit side shear is reduced with

increasing diameter regardless of length but experimental evidence suggests that the

effect is small for diameters greater than 610 mm (2 feet). Baycan (1996) used a

computer program (ROCKET 95) developed by Seidel and Haberfield (1995) to model

the behavior of a rough socket. He found that roughness and diameter have a major

effect on unit side shear. As the diameter increases radial stresses in the rock surrounding

the shaft decrease and thus the maximum unit side shear decreases as show in Figure 2.6.
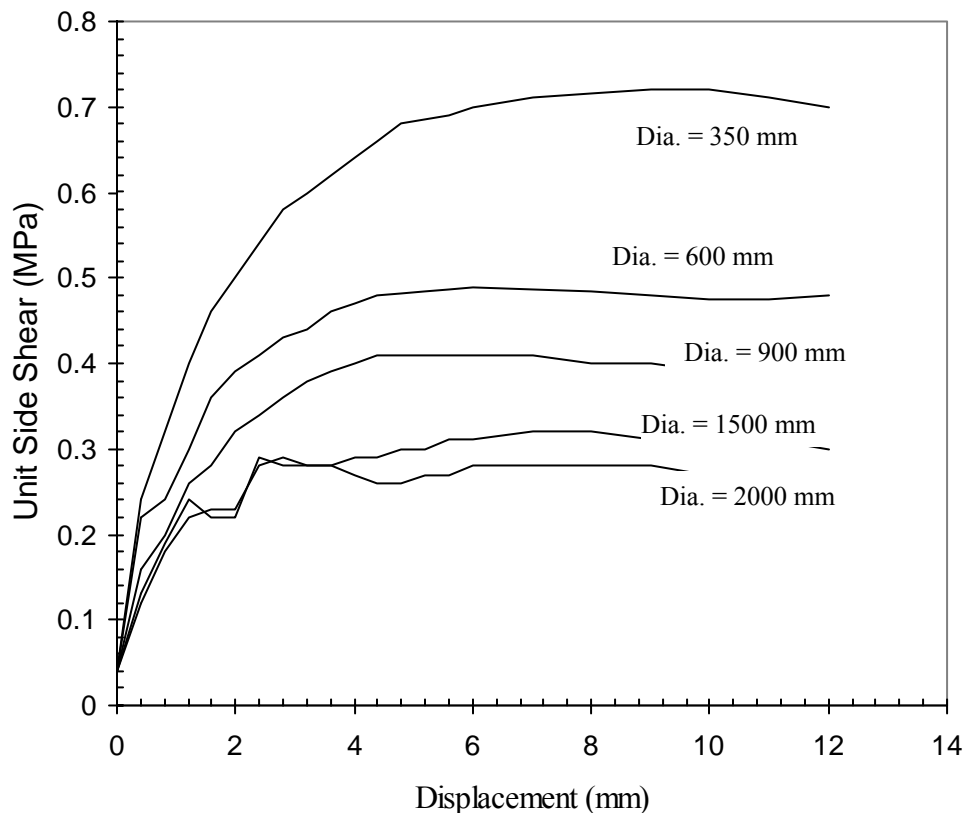
Figure 2.6- Unit side shear versus shear displacement for drilled shaft sockets in rock of
moderate roughness with q_u = 3.0 MPa (after Baycan 1996).

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2.3.8 Loading Direction

One issue directly affecting results from Osterberg load cell tests is the issue of
loading direction since unit side shear values are often determined from the portion of the
shaft that is loaded upwards while the actual field loading is generally downwards. Data
by Kulhawy and Phoon (1993) shows no significant difference in the unit side shear as a
function of loading direction, as has been noted previously by Rowe & Armitage (1984).
Ogura (1996) tested three, 1.2 m (3.9ft) diameter shafts in soft soil in Osaka, Japan one
using top-down loading and two loaded with an O-cell^{TM} placed near the base of the
38.5m (126.3 ft) shafts. The measured unit side shear was the same although the shafts
tested with the O-cells^{TM} failed in end bearing and the side shear was not fully mobilized.
Shi (2002) used a finite element model (ABAQUS) to compare the effects of loading
direction on the load taken up in side shear. Shi found that, in soil with a modulus several
orders of magnitude less than that of the concrete, there was only a slight difference in
the predicted side load with the O-cell^{TM} type (upward) loading being slightly less than
for top-down loading. However, Shi found that the difference between loads taken up by
side shear for top-down and O-cell^{TM} loading became more pronounced for rock socketed
shafts as shown in Figure 2.7, with the difference in the side load for the two methods
increasing with increasing modulus of the rock. The O-cell^{TM} loading produced
conservative values for side shear in all cases but Shi's finite element model showed the
need for further research comparing top-down loading and loading from the bottom of the
shaft using the O-cell^{TM}.
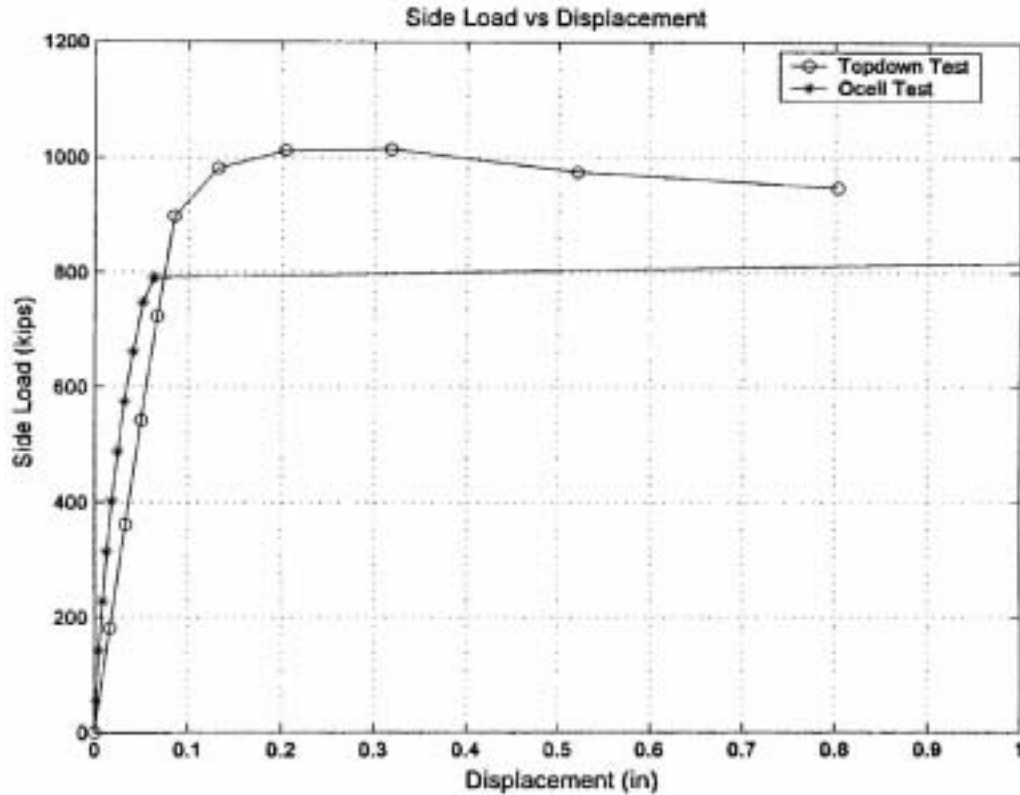
Figure 2.7- Comparison of side load versus displacement curves for top-down and O-cell^{TM} loading based on finite element analyses (from Shi 2002).

2.4 Analytical methods

Because of the difficulty in addressing all of the factors affecting the unit side shear capacity of drilled shafts using empirical methods, a number of analytical methods have been developed to predict capacity as a function of these factors. Analytical methods proposed by Kodikara et al. (1992), McVay et al. (1992), and O'Neill and Reese (1999) attempt to model the soft rock shaft interface by considering interface roughness, cleanliness of interface, initial lateral concrete pressure, properties of the weak rock such as angle of internal friction ϕ' and cohesion c', angle of interface dilation ϕ_{rc}, and the formation stiffness E_m. The following sections describe these methods in more detail.

2.4.1 Kodikara et al.

Kodikara et al. (1992) used a rational mathematical model to account for borehole roughness, initial normal stress on the interface, and stiffness of the soft rock during interface dilation. This method is an extension of finite element analyses (elasto-plastic, c', ϕ') performed by Williams et al. (1980). In the method, f_{max} is predicted as

$$f_{max} = \alpha \cdot \beta \cdot q_u \qquad\qquad (\,2.14\,)$$

where the product $\alpha \cdot \beta$ is determined from Figure 2.8, and is a function of the uniaxial compressive strength of the rock (q_u), the initial radial pressure on the interface (σ_n), the ratio of the modulus of the rock (E_m) to q_u, and the roughness of the interface. Figure 2.8 was developed for the ratio of E_m / q_u equal to 300 which is appropriate for many weak rocks. Figures for other values of E_m / q_u are also available (Kodikara et al. 1992). The roughness of the shaft interface is determined from Table 2.2 using roughness measures defined previously in Section 2.3.1 and shown in Figure 2.4. I_{sd}, which was not previously defined, is defined as the standard deviation of the angle between face of the asperity or groove and the vertical (i_m).

Table 2.2-Characterization of borehole roughness (from Kodikara et al.1992).

Parameter	Range of Values of Sockets in Melbourne Mudstone		
	Smooth	Medium	Rough
i_m (°)	10-12	12-17	17-30
I_{sd} (°)	2-4	4-6	6-8
h_m (mm)	1-4	4-20	20-80
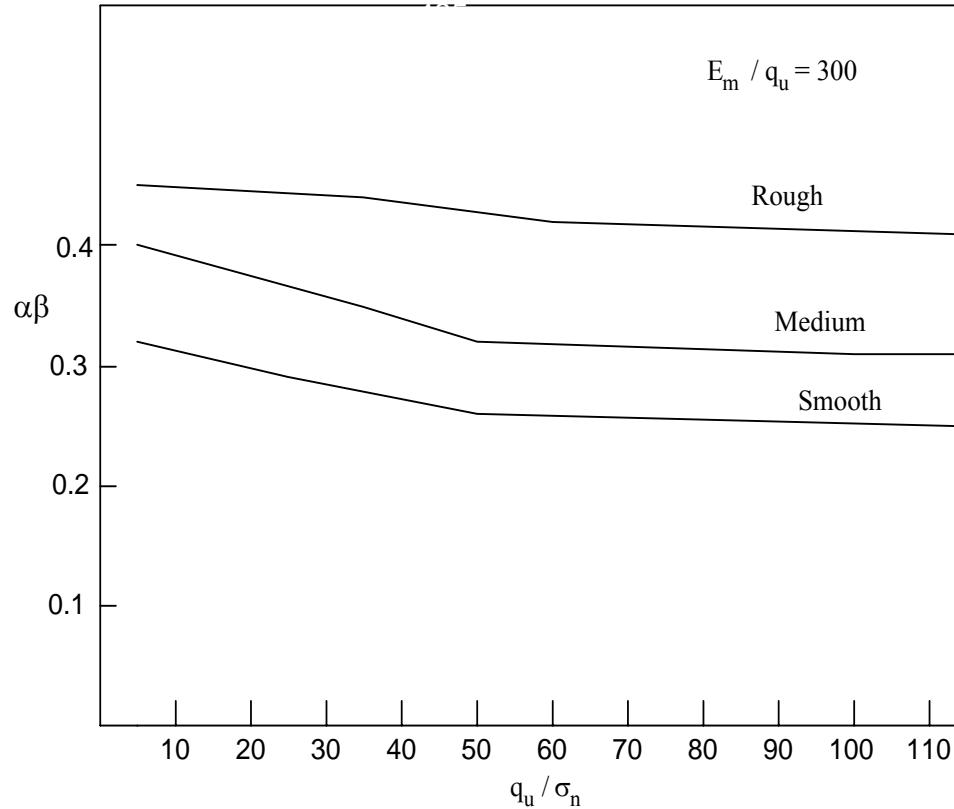
Figure 2.8- $\alpha\beta$ versus borehole roughness, where E_m is the estimated modulus or formation stiffness of the rock mass and σ_n is the initial radial pressure on the interface which can be taken as the estimated fluid pressure produced by the concrete (after Kodikara et al.1992).

2.4.2 McVay et al.

McVay et al.(1992) performed numerical analyses using a parametric finite element method to more closely examine the maximum unit side shear at the shaft-rock interface. McVay et al. found that the unit side shear is in close approximation to the cohesion value of the rock and that more than a single laboratory specimen is required to accurately determine the cohesion value. McVay et al. used both the uniaxial compression test and the splitting tensile (ASTM D3967) test to determine the cohesion

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of the rock. Using Mohr's circle and several trigonometric functions, McVay et al. found

that the maximum unit side shear was best predicted as

$$f_{max} = 0.5 \ (\ q_u \ )^{\ 0.5} \ (q_t)^{0.5} \hspace{3cm} (\ 2.15 \ )$$

Where q_t is the splitting tensile strength of the rock and f_{max}, q_u, and q_t are given in

tsf. McVay et al. found excellent agreement with the unit side shear computed by

equation 2.15 and a database consisting of 53 pullout tests and 7 load tests at 14 different

sites in Florida. The rock encountered at these test sites was a weak limestone.

2.4.3 O'Neill and Reese

O'Neill and Reese (1999) expand on finite element modeling of the Eagle Ford clay shale

performed by Hassan and O'Neill in 1993 to develop the method currently recommended

by the Federal Highway Administration (FHWA) for the calculation of f_{max} for smooth

sockets in intermediate geomaterials with unconfined compressive strengths (q_u) in the

range of 0.5 to 5.0 MPa (5 to 50 tsf).   The FHWA recommends rock sockets for drilled

shafts be designed as smooth unless they are artificially roughened. For smooth rock

sockets or sockets drilled using slurry, FHWA recommends the following expressions to

predict f_{max}

$$f_{max} = \alpha \cdot \varphi \cdot q_u \hspace{3cm} (\ 2.16 \ )$$

where α is an adhesion factor estimated from Figure 2.9, φ is a factor to account for the

presence of open joints estimated from Table 2.3, and f_{max} and q_u are given in MPa. In

Figure 2.9, σ_n is the pressure of the fluid concrete at the middle of the layer assuming the

slump of the concrete is at or above 175 mm (7 in.) and the concrete is placed at a rate of

12 m (40 feet) per hour and p_a is the atmospheric pressure in the units in which σ_n is calculated. The fluid pressure of the concrete is estimated as

$$\sigma_n = 0.65 \, \gamma_c \, Z_i \qquad\qquad\qquad (\,2.17\,)$$

where γ_c is the unit weight of concrete in kN/m^3 and Z_i is the depth to middle of layer in meters with a maximum depth of 12 m (40 feet).

Table 2.3 - Adjustment factor for soft seams and joints (from O'Neill & Reese 1999).

RQD %	φ	
	Closed Joints	Open or gouge filled Joints
100	1.00	0.85
70	0.85	0.55
50	0.60	0.55
30	0.50	0.50
20	0.45	0.45
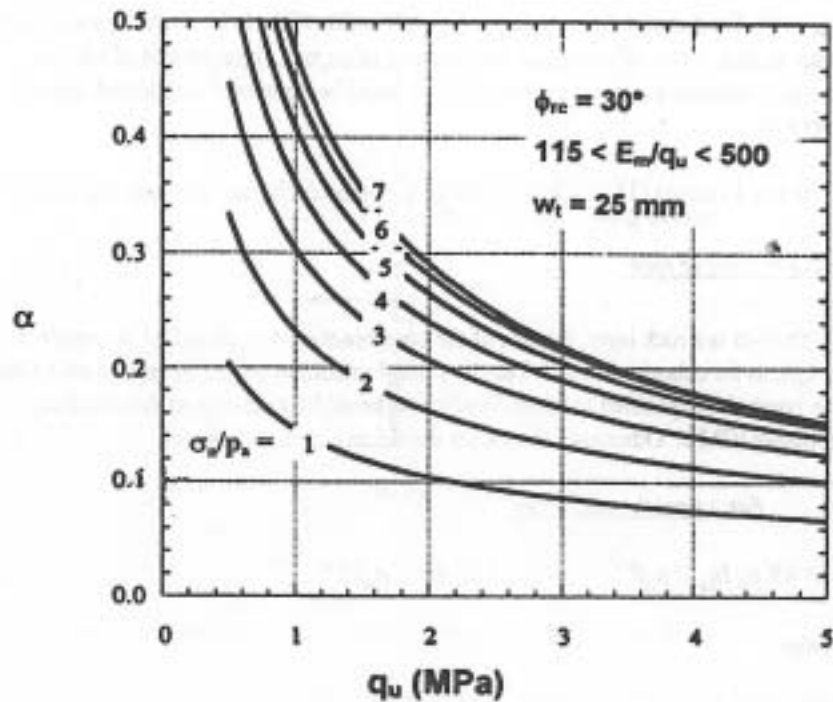
Figure 2.9- Adhesion factor α versus compressive strength q_u (from O'Neill et al.1996).

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For drilled shafts with smooth sockets and unconfined compressive strengths greater than 5.0 MPa (50 tsf) O'Neill and Reese (1999) recommend using Horvath and Kenney's (1979) method (Equation 2.5) normalized with respect to atmospheric pressure (P_a) by Carter and Kulhawy (1988). The expression takes the form;

$$f_{max} = 0.65 \, P_a \, (\, q_u / P_a \,)^{0.5} \leq 0.65 \, P_a \, (\, f'_c / P_a \,)^{0.5} \quad (2.18)$$

where f'_c is the 28-day compressive strength of the concrete and P_a is the atmospheric pressure in the units of f_{max} and q_u (P_a = 0.1013 MPa = 1.058 tsf). For rough sockets in rock, O'Neill and Reese recommended using the relationship proposed by Horvath (1983) and given in Equation 2.6.

2.5 Summary

Empirical and analytical methods for predicting the maximum side shear capacity of drilled shaft socketed into weak rock have been presented in this chapter. Empirical methods are generally based on full-scale load tests while analytical methods attempt to model the soft rock-drilled shaft interface behavior numerically.

Many of the methods for predicting ultimate unit side shear are based on empirical correlations with the unconfined compressive strength of rock cores. These relationships fall in two major groups: linear functions of q_u involving one or more coefficients and power functions of q_u involving one or more coefficients and the exponent for q_u. Many authors now believe the power-curve relationship is closer to the real case, or at least applicable over a broader range of q_u.

Other authors have related ultimate unit side shear to factors created by construction technique, properties of the rock mass, and geometry of the socket using numerical models. Linear relationships to address these additional factors were proposed by Kodikara et al. (1992) and O'Neill and Reese (1999). McVay et al. (1992) proposed a power function relationship.

CHAPTER THREE
GENERAL OSTERBERG CELL TEST METHODS AND PROCEDURES

A new test method for full-scale load testing of drilled shafts has recently been developed that uses an Osterberg load cell (O-cell^{TM}) invented by Dr. Jorj Osterberg. The Osterberg cell, along with instrumentation such as strain gages and telltales, can be used to determine end bearing and side shear capacities of drilled shafts and piles. This chapter includes a general description of the Osterberg cell load test method and the general procedure used to perform O-cell^{TM} load test. Methods for analysis and interpretation are then described followed by the procedure used to determine ultimate unit side shear values from O-cell^{TM} load test results.

3.1 Description of Osterberg cell load test

The Osterberg load cell (O-cell^{TM}) is a sacrificial, jack-like device that is used to test the axial capacity of drilled shafts. The O-cell^{TM} may be attached either to the rebar cage or a carrying frame and is usually positioned at the base of the shaft or some distance above the base as shown in Figure 3.1. As the O-cell^{TM} is expanded under hydraulic pressure, it simultaneously exerts an upward force against the portion of the shaft above the cell and an equal downward force against the portion of the shaft and/or bearing strata below the cell. The O-cell^{TM} has the advantage of being able to apply large loads on drilled shafts without the need for a large reaction system.

Dr. Jorj Osterberg, Professor Emeritus at Northwestern University developed and patented the test. The O-cell^{TM} was first used on a bored pile in 1984. The O-cell^{TM} evolved from a bellows type expansion cell to the current design which is very similar to the piston type jack commonly used on conventional load tests, except that the piston

extends downward instead of upwards (Schmertmann 1997). Approximately 400 tests

have been performed in the United States and Southeast Asia (Osterberg 1999).

LoadTest, Inc. of Gainesville, Florida is currently the exclusive distributor of the O-

cell[TM] and provides installation and test support services.

O-cell[TM] test capacities have increased steadily over recent years. A 1993 test

performed for the Kentucky Transportation Cabinet for a bridge across the Ohio River at

Owensboro, reached a load of 54 MN (6000 + tons) in each direction (Goodwin 1993).

In 1997, a test for the Florida Department of Transportation (FDOT) across the

Apalachicola River reached a load of 133 MN (14950 tons) in both directions

(Schmertmann 1998). The Florida test used three 864 mm (34 in.) diameter O-cells[TM] in

a 2.75 m (9.0 ft.) diameter shaft socketed 13.7 m (50 ft.) into limestone. On January 30,

2001, an O-cell[TM] load test was performed in Tucson, Arizona for the Arizona

Department of Transportation (ADOT) that reached a load of 151 MN (17,000 tons) in

both directions. The O-cell[TM] load test was performed on a 2.43 m (8.0 ft) diameter shaft

41.3 m (135.5 ft) deep. Three 864 mm (34 in.) diameter O-cells[TM] were installed 8.7 m

(28.5 ft) above the base of the shaft. The drilled shaft was constructed by drilled shaft

contractors, Anderson Drilling (Lakeside, CA) and Case Foundation (Rosedelle, IL).

Anderson Drilling used "Big Stan", the world's largest truck-mounted drill rig to

excavate the shaft. The available O-cell[TM] sizes and capacities are shown in Table 3.1.
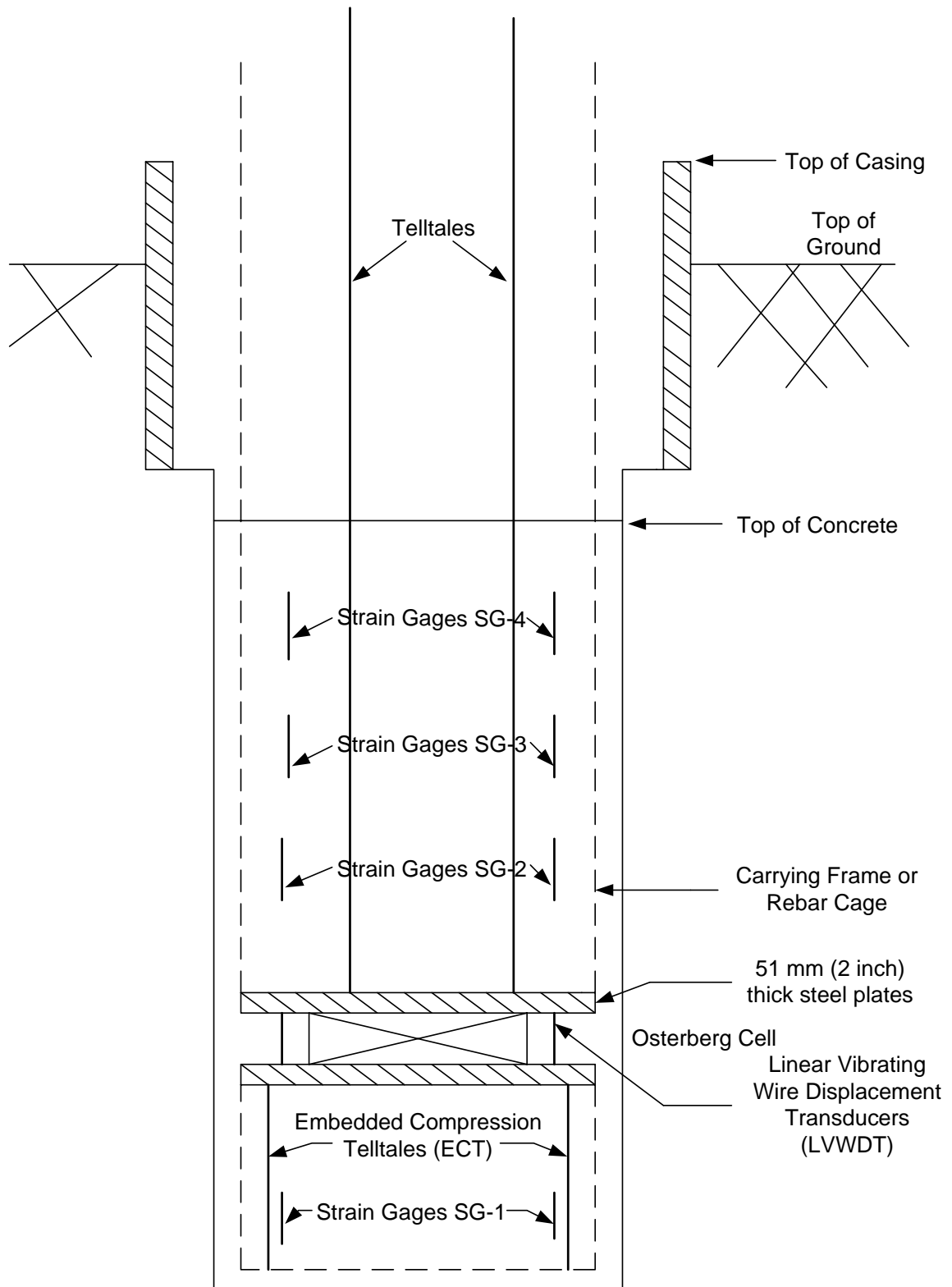
Figure 3.1-Schematic of typical Osterberg cell load test.

Table 3.1-Available O-cell^{TM} sizes and capacities.

Nominal Diameter		Nominal Capacity	
(mm)	(in)	(MN)	(tons)
230	9	1.8	200
330	13	3.6	400
540	21	8.9	1000
660	26	16.0	1800
870	34	27.0	3000

3.2 Osterberg Load Test Procedure

O-cell^{TM} load tests are generally performed by pressurizing the O-cell^{TM} in increments and monitoring displacements at the top and bottom of the shaft and strains at various points along the shaft. The O-cell^{TM} is usually pressurized following the ASTM Quick Test Method (ASTM D1143), although other methods may be used. The Quick Test Method stipulates that small increments of load be applied every four minutes. As the O-cell^{TM} expands, the side shear (f_s) developed above the O-cell^{TM}, between the concrete of the shaft and the walls of the rock socket, serves as the reaction to develop the end bearing (q) below the O-cell^{TM}, or end bearing and side shear if the O-cell^{TM} is set some distance above the bottom of the rock socket. Simultaneously, the end bearing and any side shear derived below the O-cell^{TM} serve as the reaction to apply load to the upper part of the shaft above the O-cell^{TM}. By loading in this manner, load-displacement curves for both the upper and lower portions of the shaft are obtained simultaneously as shown in Figure 3.2. The test is continued until the shaft fails in end bearing or side shear, the capacity of the O-cell^{TM} is exceeded, or in some cases, until two to three times the design load is achieved. The objective of the O-cell^{TM} test is to position the O-cell^{TM} so that failure in side shear and end bearing occurs simultaneously.

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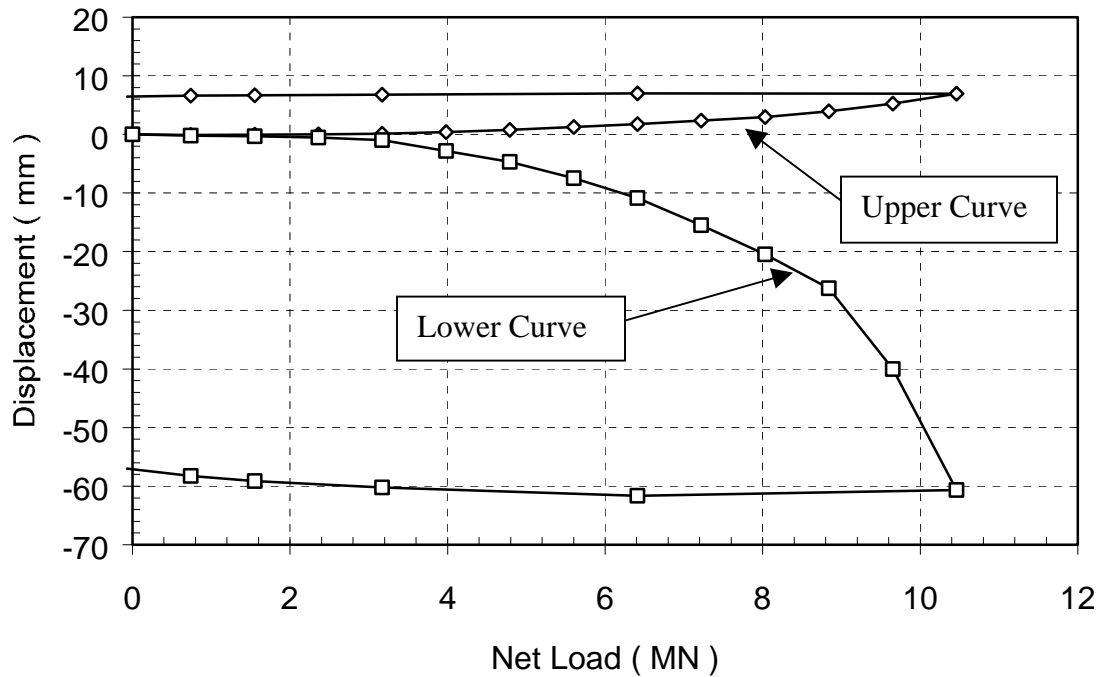
Figure 3.2-Typical Osterberg load test load-displacement curves.

3.3 Instrumentation and Data Collection

Displacements and strains that take place during an O-cell[TM] test are typically measured by electronic gages connected to a computerized data acquisition system. Figure 3.1 shows a schematic indicating the various types of instruments commonly used. The opening or extension between the top and bottom plates of the O-cell[TM] is measured by two, or sometimes three Linear Vibrating Wire Displacement Transducers (LVWDTs) attached to the bottom plate of the cell. Upward movement of the top of the shaft is measured using dial gages or Linear Voltage Displacement Transducers (LVDT) mounted on a reference beam set over the top of the shaft. The upward movement of the O-cell[TM] is measured using a pair of steel telltales that extend from the top of the O-cell[TM] to the top of the shaft. The telltales also provide for measurement of the

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compression of the shaft if the displacement of the top of the shaft is monitored. The

downward movement of the bottom plate is determined by subtracting the upward

movement of the top of the O-cell^{TM} from the total extension of the O-cell^{TM}. The

reference beam is in turn monitored by a surveyor's level. Typical instrumentation of the

top of shaft is shown in Figures 3.3 and 3.4. The load applied by an O-cell^{TM} is calibrated

versus hydraulic pressure before installation and pressure to the cell is measured during

the test using a Bourdon gage or vibrating wire pressure transducer.

Telltales with LVDTs
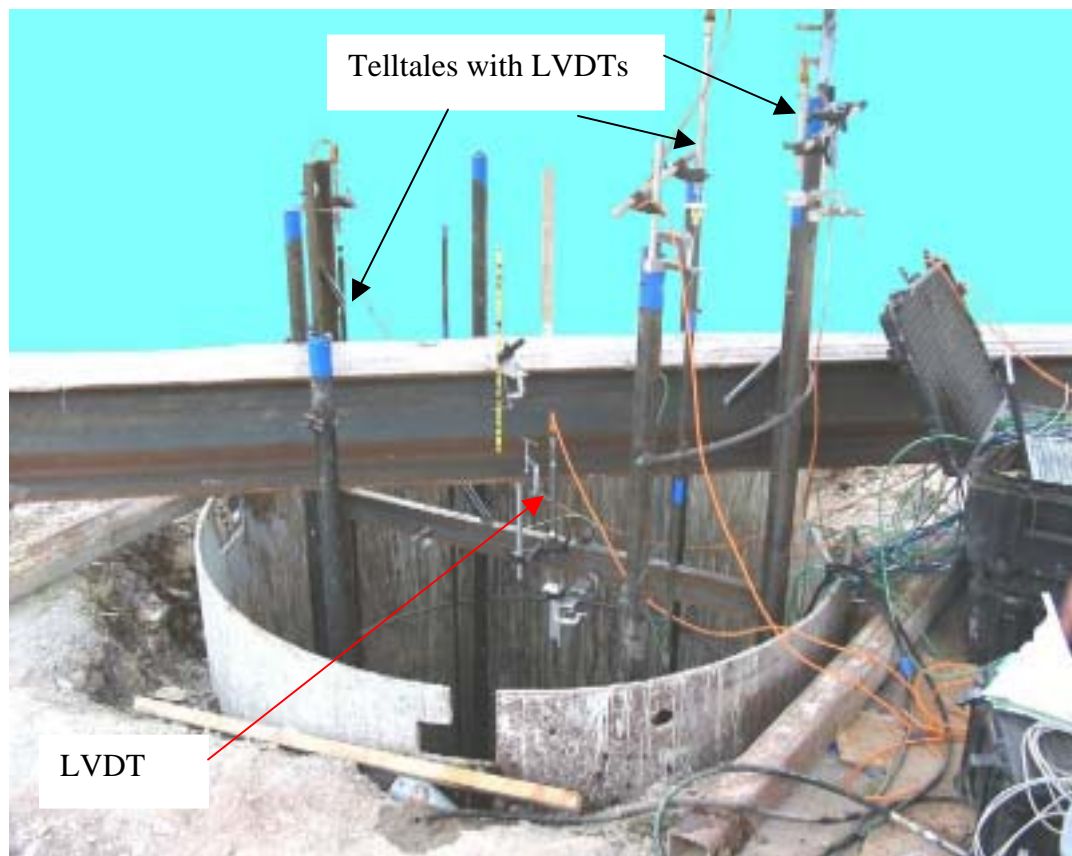
LVDT

Figure 3.3- Typical instrumentation at the top of a drilled shaft with telltales monitored
with LVDTs to determine shaft compression and a LVDT attached to a
reference beam to monitor movement of the top of the shaft. The LVDT on
the middle telltale is monitoring movement of the bottom plate of the O-
cell^{TM} assembly. The LVWDTs used to monitor top of shaft movement are
not visible.

Figure 3.4-Typical instrumentation at the top of a drilled shaft, telltales monitored with digital dial gages. The 25 mm (1 in) stroke dial gages are used to monitor compression of the shaft and are shown. The 100 mm (4 in) stroke dial gages are used to monitor top of shaft movement and are hidden behind the reference beam, although the magnetic base of the right one is visible (from Loadtest).

In addition to the basic instrumentation described above, strain gages are often installed at various positions along the shaft as shown in Figure 3.1 to facilitate determination of load transfer along the length of the shaft. Two types of embedment strain gages are commonly used in drilled shafts to monitor strain and shaft compression of the concrete. The first type is a concrete embedment stain gage with large flanges at both ends to provide anchorage into the concrete as shown in Figure 3.5. The second type of gage is a "sister bar" consisting of strain transducer mounted on the central portion of a length of reinforcing steel as shown in Figure 3.6 and 3.7. Both concrete

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embedment gages and sister bars are available with vibrating wire and fiber optic strain

sensors. Sister bars are, at times, used with inexpensive foil resistance type strain gages,

although these types of gages are not nearly as robust as vibrating wire or fiber optic

gages. The advantage of using vibrating wire strain gages over more conventional electric

resistance gages is the sensor output, which is frequency rather than a voltage or

resistance. The frequency output is easier to transmit over long cables and is unaffected

by voltage drops that may be caused by corrosion, moisture, or temperature effects. The

frequency signal is also not affected by changes in the length of the sensor cables (Hayes

2002). Fiber optic gages have similar advantages and are not affected by temperature

("self compensating").
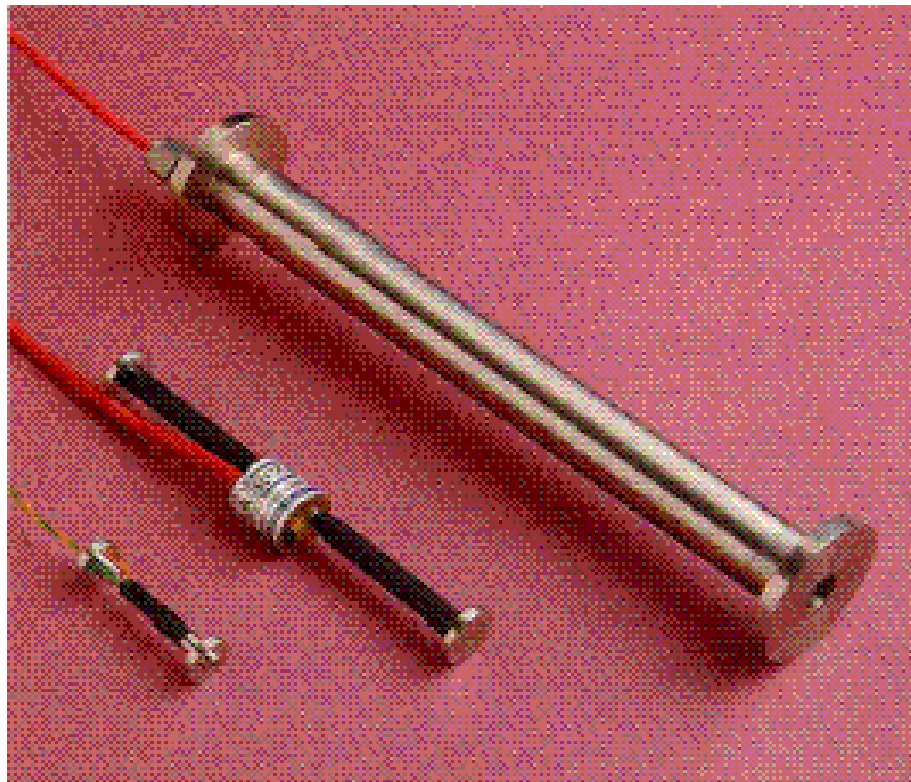
Figure 3.5- Embedment strain gages (from Geokon).

Figure 3.6- Sister bars used to monitor strains within drilled shafts.

Sister Bar

Figure 3.7- Sister bar mounted on rebar cage.

Compression of the shaft concrete may also be measured by using an Embedded Compression Telltales (ECT) as shown in Figures 3.8 and 3.9. The ECT assemblies generally consist of a 13 mm (1/2 inch) steel casing with an inner 6 mm (1/4 inch) steel rod. A linear vibrating wire displacement transducer is attached to the 6 mm steel rod to monitor displacements.

Figure 3.8- Embedded compression telltale (ECT) gage mounted in rebar cage.
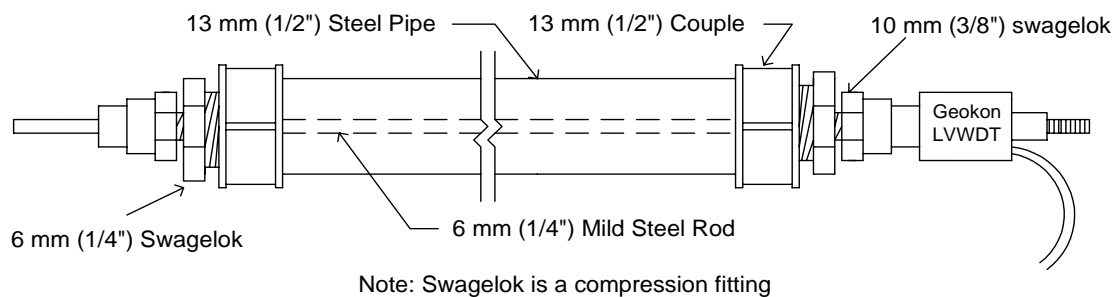
Figure 3.9- Schematic of embedded compression telltale.

3.4 Basic Interpretation of O-cell[TM] Tests

The O-cell[TM] test provides two separate load-displacement curves as shown in Figure 3.10. One curve, referred to as the "upper" curve, describes the upward displacement of the shaft above the O-cell[TM] versus applied load. This curve represents the resistance provided by side shear above the O-cell[TM] plus the buoyant weight of the shaft above the O-cell[TM] as a function of displacement (Schmertmann 1998). The other, "lower" curve describes the downward displacement of the shaft below the O-cell[TM] as a function of the load, which represents the resistance derived from end bearing plus any upward side shear between the O-cell[TM] and the base of the shaft.
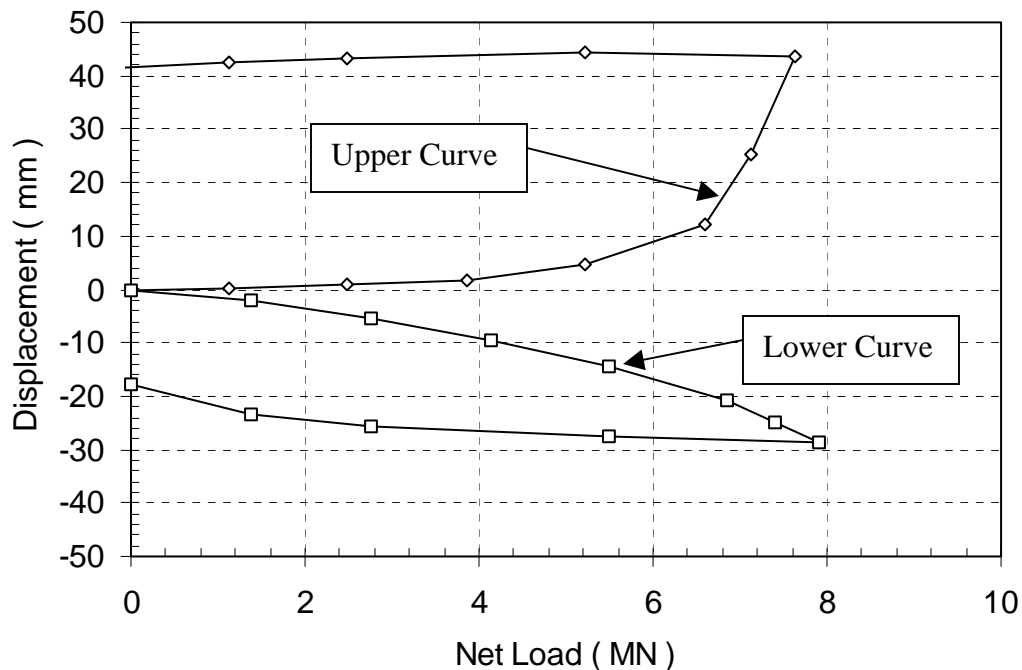
Figure 3.10- Typical Osterberg cell load-displacement curves.

Based on the measured upward and downward responses from an O-cell[TM] test, an equivalent "top-down" load-displacement curve can be developed using a procedure described by Osterberg (1998). The procedure consists of first picking a value of

displacement and determining the corresponding loads from the upward and downward response curves. The loads for the upward and downward response curves are then added together and plotted versus the selected value of displacement. This procedure is illustrated in Figure 3.11 for a displacement of 10 mm. In this case the load from the upper curve is 6.4 MN and the load from the lower curve is 4.2 MN, which results in an equivalent top-down load of 10.6 MN. Another value of displacement is then selected and the process repeated until a complete top-down load-displacement curve is generated.

A problem often arises with the equivalent top-down procedure in that often the shaft will fail in upward side shear before the end bearing is fully developed (or vice-versa). When this occurs, there is not enough data on the lower curve to define loads at larger displacements and the equivalent top-down load-displacement curve cannot be completely generated. To remedy this problem, Osterberg (1998) has proposed applying a hyperbolic extrapolation to the downward load-displacement curve to generate enough points to complete the equivalent top-down load-displacement curve. This process may be reversed if the downward portion of the shaft fails before the upward portion of the shaft.

Several basic assumptions must be made in order to construct the equivalent top-down load-displacement curve. These assumptions include:

1. The load-displacement curve resulting from the upward displacement of the top of the shaft is identical to the downward displacement of the shaft in a conventional, top-down compression load test.

Upward and Downward Load Displacement Curves from O-cell Test

Equivalent "Top-Down" Curve
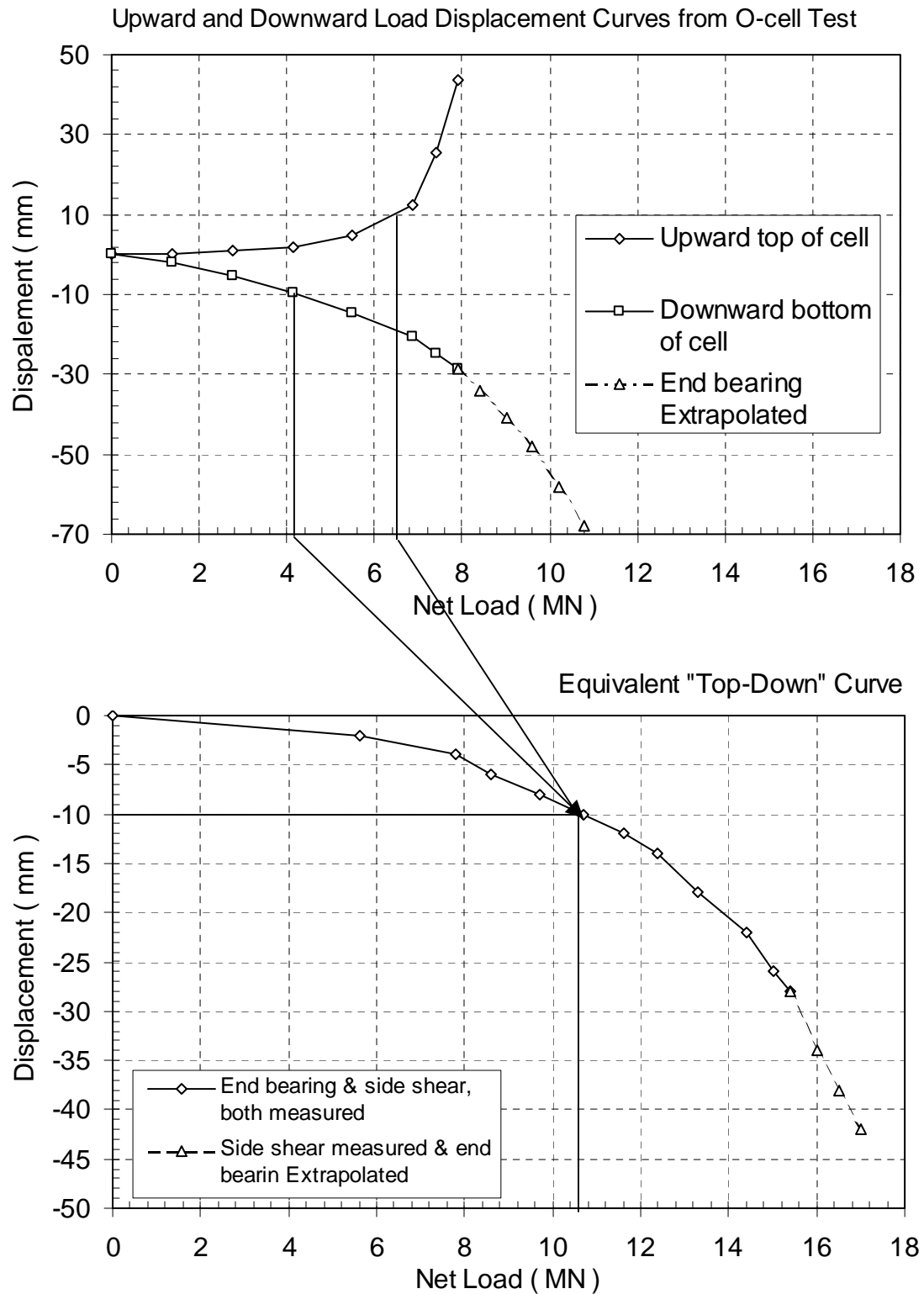
Figure 3.11- Construction of equivalent "top-down" load-displacement curve from upwards and downwards load-displacement curves from O-cell^{TM} test.

2. The load-displacement curve resulting from the downward displacement of the bottom of the O-cell[TM] is identical to the downward bottom-of-shaft displacement in a conventional top-down load test.

3. The compression of the shaft is considered negligible.

Based on finite element analyses and a search of literature, engineers at LoadTest Inc. and researchers at the University of Florida and elsewhere have indicated that the O-cell[TM] test produces slightly lower values of the ultimate unit side shear than obtained from conventional top-down load tests. Data from Kulhawy and Phoon (1993) shows no significant difference as a function of loading direction; similar observations were noted previously by Rowe & Armitage (1984). Ogura (1996) tested 1.2 m (3.9ft) diameter shafts in soil in Osaka, Japan using both top-down loading and loading with an O-cell[TM] placed near the base of the 38.5m (126.3 ft) shafts. The measured unit side shear was similar for both tests. However, the shafts tested with the O-cells[TM] failed in end bearing prior to side shear being fully mobilized.   Engineers at Loadtest Inc. compared the predicted equivalent displacement to measured displacement for drilled shaft load tests performed in Japan by Kisida et al. (1992) and Ogura et al. (1995). The ratio of predicted equivalent displacement to measured displacement, averaged 1.03 with a coefficient of variation of 9.4 percent.

Shi (2002) used a finite element model (ABAQUS) to compare the effects of loading direction on the load taken up in side shear. Shi found that for a soil with a modulus several orders of magnitude less than that of the concrete, there was only a slight difference in the predicted side loads from top-down and bottom-up loading and that the Osterberg cell load test produced results that were slightly conservative. Shi also found

that the difference between load taken up by side shear in top-down and bottom-up

loading is more pronounced for rock socketed shafts where the modulus of the rock is

similar to or greater than that of the concrete. Shi further found that the difference in the

side load for top-down and bottom-up loading increases with increasing modulus of the

rock. The Osterberg cell load test was found to produce conservative values for unit side

shear, but Shi's finite element analyses demonstrate the need for further research

comparing top-down loading and shafts loaded from the bottom-up using the Osterberg

load cell.

To construct the equivalent top-down load-displacement curve from the O-cell^{TM}

test results, the drilled shaft is initially assumed to behave as a rigid body and the real

elastic compression that is part of the movement data obtained from an O-cell^{TM} load test

is included in the construction of the equivalent top-down load-displacement curve.

However, the elastic compression in the equivalent top-down test always exceeds the

elastic compression in an O-cell^{TM} load test. This assumption is based on the premise

that soil strength typically increases with depth and as the load is dissipated through side

shear the elastic compression decreases. In a top-down test, load is applied where the soil

unit side shear is the weakest and the elastic compression is greatest whereas in the O-

cell^{TM} test the load is applied at the base of the shaft were the unit side shear is the

greatest and the elastic compression is the least. Loadtest presents an approximate

solution for determining the additional elastic compression of a top-down loaded shaft in

their procedures for the construction of equivalent top-down load-displacement curves

(August 2000). The procedure consists of first assuming a load distribution along the

shaft and then determining the elastic compression or deflection (δ_{OLT}) for a shaft

loaded with either one or two Osterberg load cells and the elastic deflection (δ_{TLT}) of an

equivalent top-down loaded shaft. The additional elastic compression or deflection ($\Delta\delta$)

is determined as

$$\Delta\delta = \delta_{TLT} - \delta_{OLT} \qquad\qquad (3.1)$$

The additional elastic compression or deflection $\Delta\delta$ is added to the "rigid" top-down load

displacement curve previously constructed to obtain the final corrected equivalent top-

down load-displacement curve as shown in Figure 3.12.
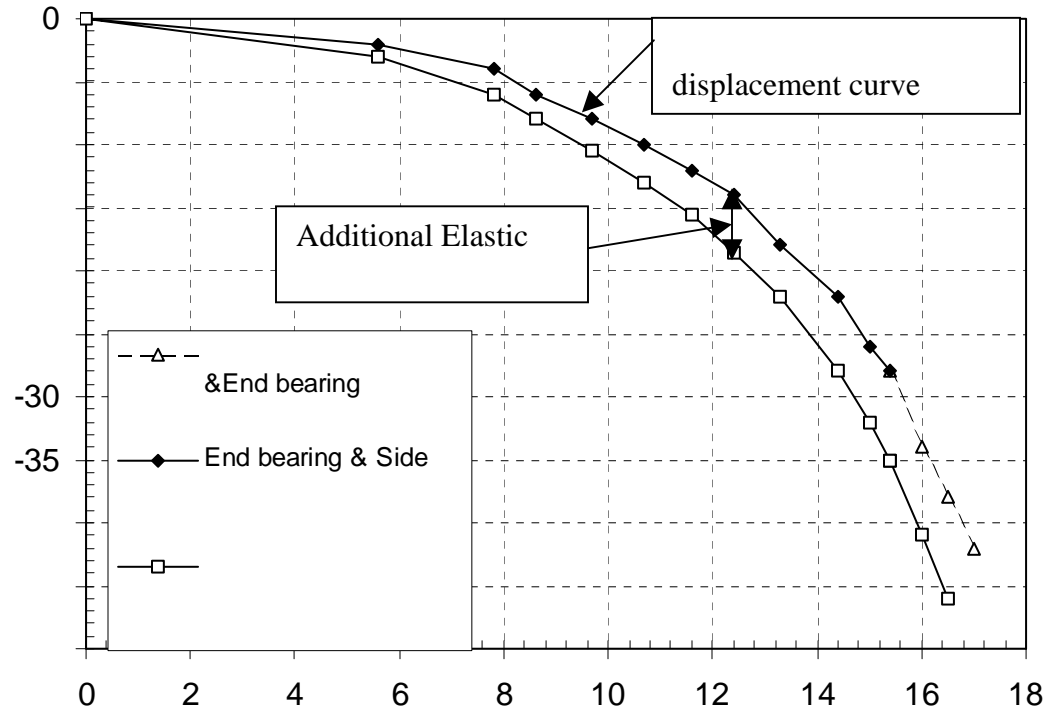
Figure 3.12- Equivalent top-down load-displacement curve adjusted for additional elastic
       compression.

3.5 Determination of Creep Limit

The creep limit is generally defined as the load at which further loading of the

shaft will cause displacement of the shaft to occur freely. The creep limit is determined

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by plotting the displacement that may occur over the time interval between 2 to 4 minutes after applying a load, while the load is maintained constant as shown in Figures 3.13a and b. A break in the curve of displacement versus load indicates the creep limit-the point at which displacement begins to accelerate under constant load. In Figure 3.13 the creep limit for the upper portion of the shaft occurs at a load of 4.7 MN. At an applied load of 4.7 MN the upper segment of the shaft has displaced a total of 3.4 mm as determined from the overall load-displacement curves as shown in Figure 3.14. The creep limit for the combined end bearing and side shear from the lower portion of the shaft occurs at a load of 1.6 MN and a total displacement of 3.3 mm.

In a top-down loaded shaft, creep cannot begin to occur freely until the overall load exceeds a combined creep limit. Although the creep limit is generally defined as the load at which further loading of the shaft will cause displacement of the shaft to occur freely, it is usually determined as the displacement corresponding to the limiting load. A conservative approach would be to set the combined creep limit equal to the lesser displacement for the creep limits determined in Figures 3.13 and 3.14. However, Loadtest recommends setting the combined creep limit to be the equivalent top-down load where the total displacement is equal to the larger of the displacements determined from Figures 3.13 and 3.14. They believe this procedure more nearly matches the actual case. Application of the recommended procedure to determine the combined creep limit for the data shown in Figures 3.13 and 3.14 for an equivalent top–down load-displacement curve is a follows. For a loading of 4.7 MN the upper segment of the shaft displaces 3.4 mm while for a loading of 1.6 MN the lower segment of the shaft displaces 3.3 mm as determined from Figure 3.14. The maximum displacement of 3.4 mm is

plotted on the "rigid" equivalent top-down load-displacement curve as shown in Figure

3.15 to determine the combined creep limit of 7.9 MN. If a creep limit cannot be

determined for either the upper or load displacement curve, the maximum displacement

recorded is used to determine the combined creep limit for the equivalent top-down load-

displacement curve.

3.6 Determination of Load Transfer (t-z) Curves

Load transfer curves showing the distribution of force in the shaft versus depth

may be obtained from the strain gage data in a manner identical to that used for top-down

load tests. Several levels of strain gages are typically installed in the shaft at strata

changes or at other points of interest. The modulus of the shaft concrete is determined

from concrete cylinders prepared from the original shaft concrete and tested on the day of

the O-cell^{TM} test. The modulus of the concrete in the shaft (E_c) may be calculated by

applying the ACI formula to the compressive strength of concrete (f'_c)

$$E_c = 4700 \, f'^{0.5}_c \qquad\qquad\qquad (3.2)$$

where E_c is the modulus of the concrete in MPa and f'_c is the 28-day compressive

strength of the concrete in MPa. Equation 3.2 is applicable for concrete with a unit

weight (γ_c) greater than or equal to $14 \, kN/m^3$ and less than or equal to $25 \, kN/m^3$. In

English units the expression is given as:

$$E_c = 57000 \, f'^{0.5}_c \qquad\qquad\qquad (3.3)$$

where E_c and f'_c are given in psi and Equation 3.3 applies to concrete with a unit weight

γ_c greater than or equal to 90 pcf and less than or equal to 155 pcf.
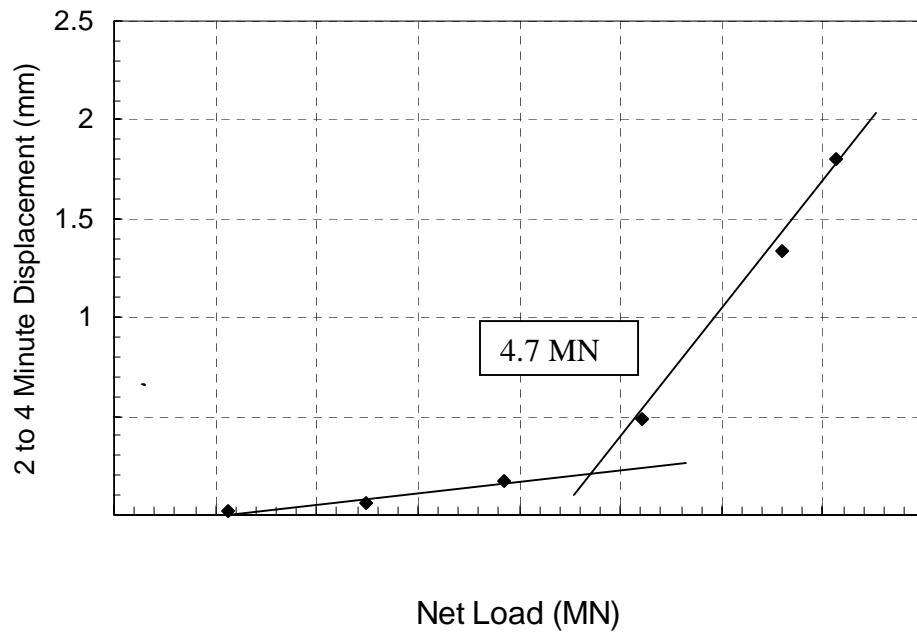
a. Upper portion of shaft.
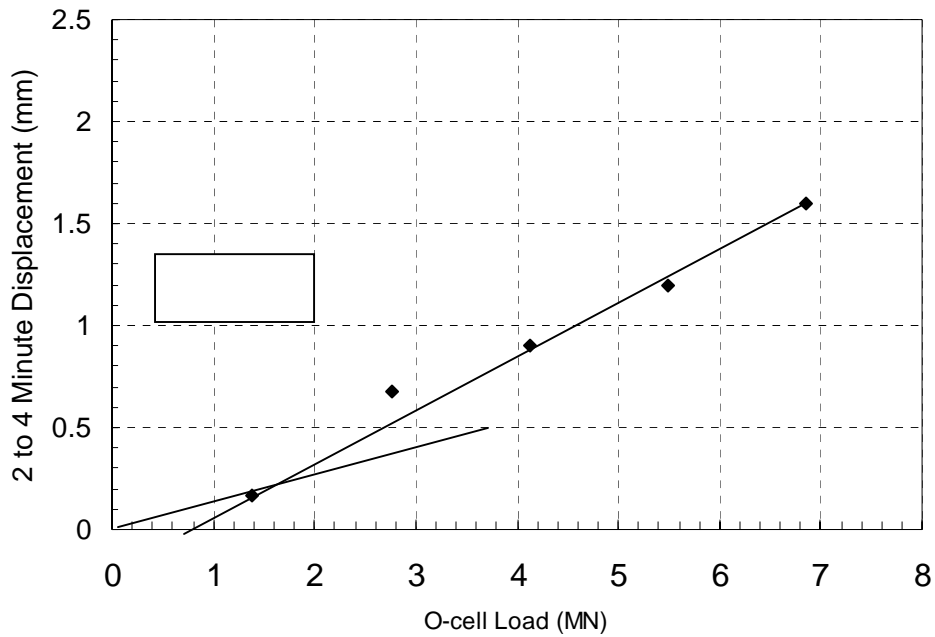
b. Lower portion of shaft.

Figure 3.13- Creep displacements from O-cell^{TM} load tests as a function of applied load: (a) upper portion of shaft and (b) lower portion of shaft.
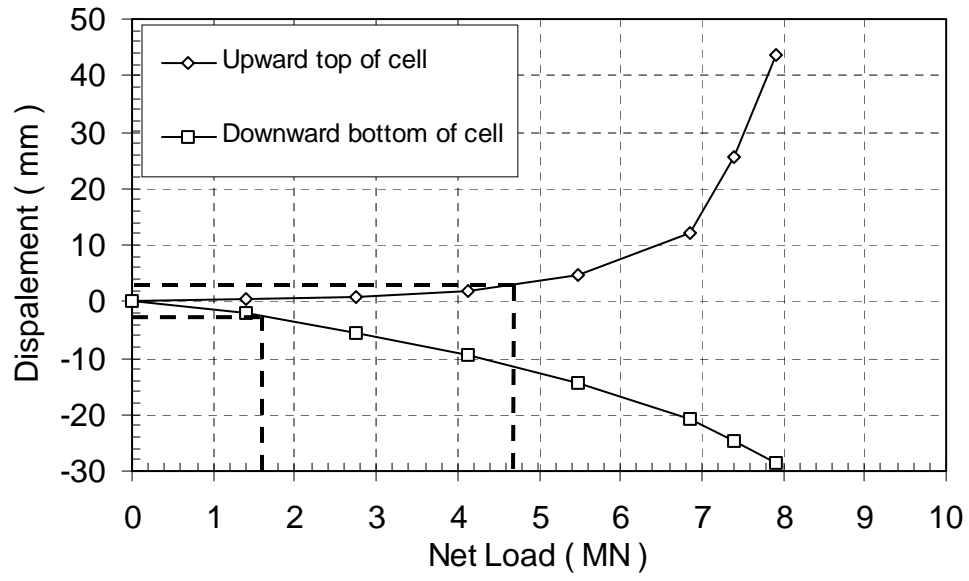
Figure 3.14- Osterberg cell load-displacement curves showing creep-limit displacements.
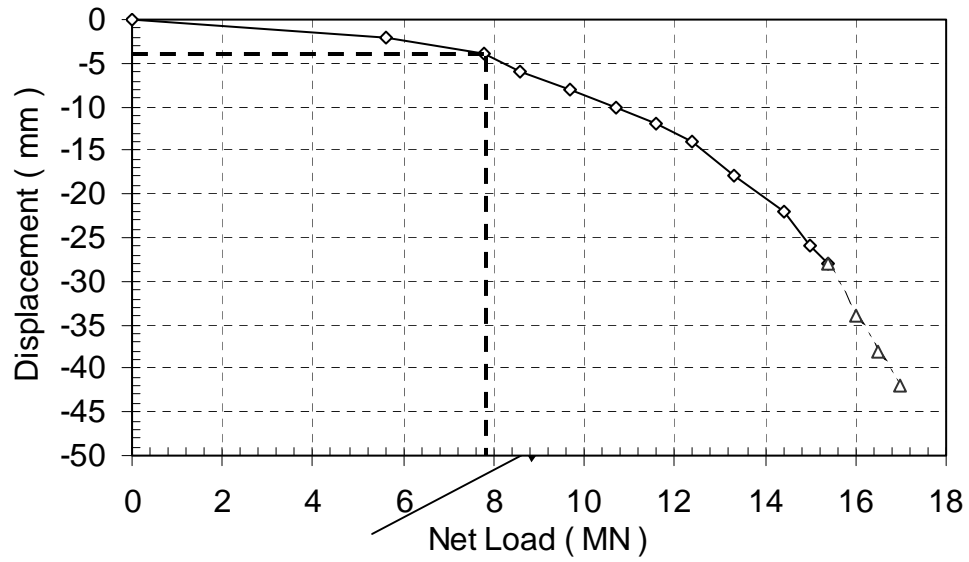
Figure 3.15- Equivalent top-down load-displacement curve with creep limit.

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Knowing the modulus of the concrete, the area of the steel, and the area of

concrete in the shaft, an "equivalent" shaft modulus (E_p) can be determined as

$$E_p = \frac{E_c A_c + E_s A_s}{A_p} \qquad (3.4)$$

where A_c is the area of the concrete, A_s is the area of the reinforcing steel, A_p is the total

cross sectional area of the drilled shaft at the point of interest, and E_s is the modulus of

the reinforcing steel. The diameter of the drilled shaft may be obtained by using either a

mechanical caliper or sonar methods. Sonic calipers are also available and a new method

developed in Australia uses a laser. Using the "equivalent" shaft modulus calculated in

Equation 3.4, the average axial stress at a given elevation (σ_i) can be computed as

$$\sigma_i = E_{pi} * \varepsilon_{axial\text{-}i} \qquad (3.5)$$

where $\varepsilon_{axial\text{-}i}$ is the axial strain determined from strain gages within the shaft.

Strain gages are generally installed at various elevations along the length of the

shaft with two to four strain gages installed at each elevation of interest. Typically the

average strain at elevation i is determined from the average of the strain gages at that

elevation. The axial force at elevation i (F_i) is then computed as

$$F_i = \sigma_i A_{pi} = A_{pi} E_{pi} \varepsilon_{axial\text{-}i} \qquad (3.6)$$

where A_{pi} is the shaft area at elevation i. The distribution of axial force with elevation

or depth can then be calculated and plotted as shown in Figure 3.16. In the figure, the

notation used to indicate each load is as follows: 1L-4 denotes the first loading event,

load increment number 4. The curve for 1L-4 indicates that 5.5 MN is applied at the

elevation of the Osterberg cell. The load at the level 1stain gages for this applied load is

about 2.35 MN. This means that 3.15 MN of axial load has been taken up in side shear

between the O-cell^{TM} and SG-1 (5.5 – 2.35 = 3.15). The maximum applied load for an

Osterberg cell load test occurs at the elevation of the O-cell^{TM}. This is contrary to a top-

down load test where the maximum load is applied at the top of the shaft as seen in

Figure 3.17.
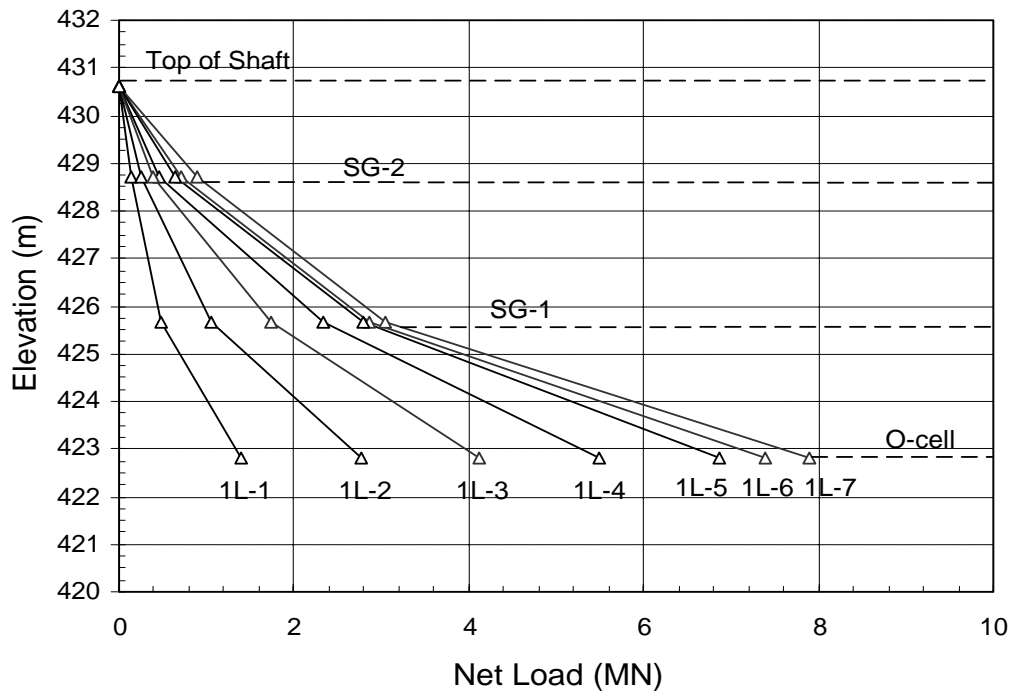
Figure 3.16- Load distribution curves determined from strain gage data.
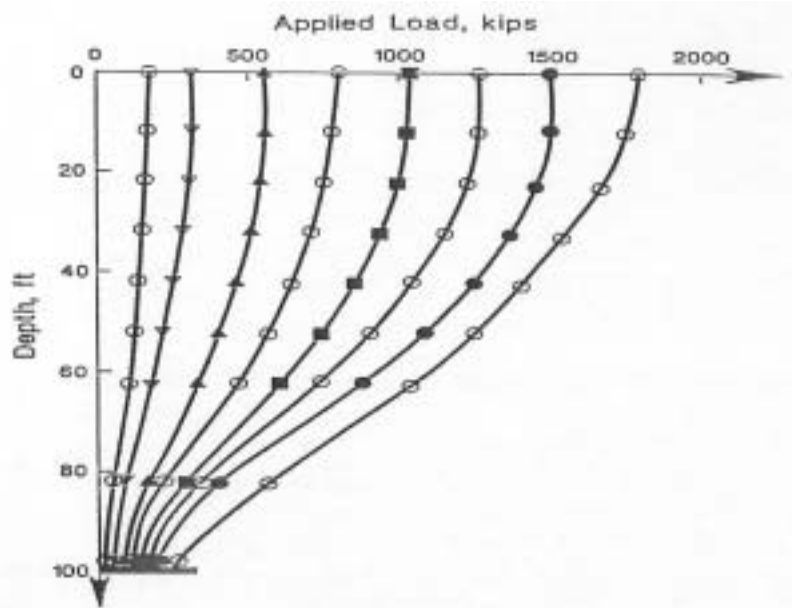
Figure 3.17- Typical load distribution from a top-down load test on a drilled shaft (from Reese 1984).

3.7 Determination of Unit Side Shear

Once the distribution of load along the shaft is known, the value of the average unit side shear for a particular segment of the shaft is calculated as

$$f_S = \Delta F_i / (\text{Shaft Perimeter} * \Delta z_i) \qquad (3.7)$$

where f_S is the unit side shear for the shaft segment, ΔF_i is the change in axial force over the length of the shaft segment, and Δz_i is the length of the shaft segment. This procedure is shown graphically for a conventional top-down load test in Figure 3.18 where Q_b and Q_S are axial loads resisted by end bearing and side shear, respectively. The values of the average unit side shear for each segment are generally plotted versus the O-cell^{TM} displacement as shown in Figure 3.19 to help determine if the maximum unit side shear was achieved.

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Shi (2002) used a finite element model to compare the maximum unit side shear that would be determined from a top-down load test with that of the bottom loaded Osterberg cell load test for a test shaft constructed in Wilsonville Alabama as shown in Table 3.2. The 812mm (32 in) diameter shaft was socketed 5.6 meters (18.5 ft) into shale with an unconfined compressive strength of 8.96 MPa (93.6 tsf). The shaft was tested by Loadtest Inc. on February 9, 1994 to a maximum load of 4.75 MN (534 tons). At this load the upward displacement was 17 mm (0.66 in) and the downward displacement was 61 mm (2.384 in). Although overall the average unit side shear along the length of the shaft is similar for the top-down load test compared to the O-cell^{TM} load test, the O-cell^{TM} load test method has a tendency to have higher values of unit side shear closer to the bottom of the shaft or closer to the O-cell^{TM}.

3.8 Summary

A new test method for full-scale load testing of drilled shafts and piles has been presented in this chapter. This method consists of placing an Osterberg load cell (O-cell^{TM}) at or near the base of a drilled shaft or pile to test the axial capacity of the shaft or pile. The Osterberg load cell along with instrumentation such as strain gages and telltales can be used to determine end bearing and side shear capacities of drilled shafts and piles. This chapter includes a general description of the Osterberg cell load test method and the general procedure used to perform O-cell^{TM} load tests. Methods for analysis and interpretation are then described followed by the procedure used to determine ultimate unit side shear values from O-cell^{TM} load test results.
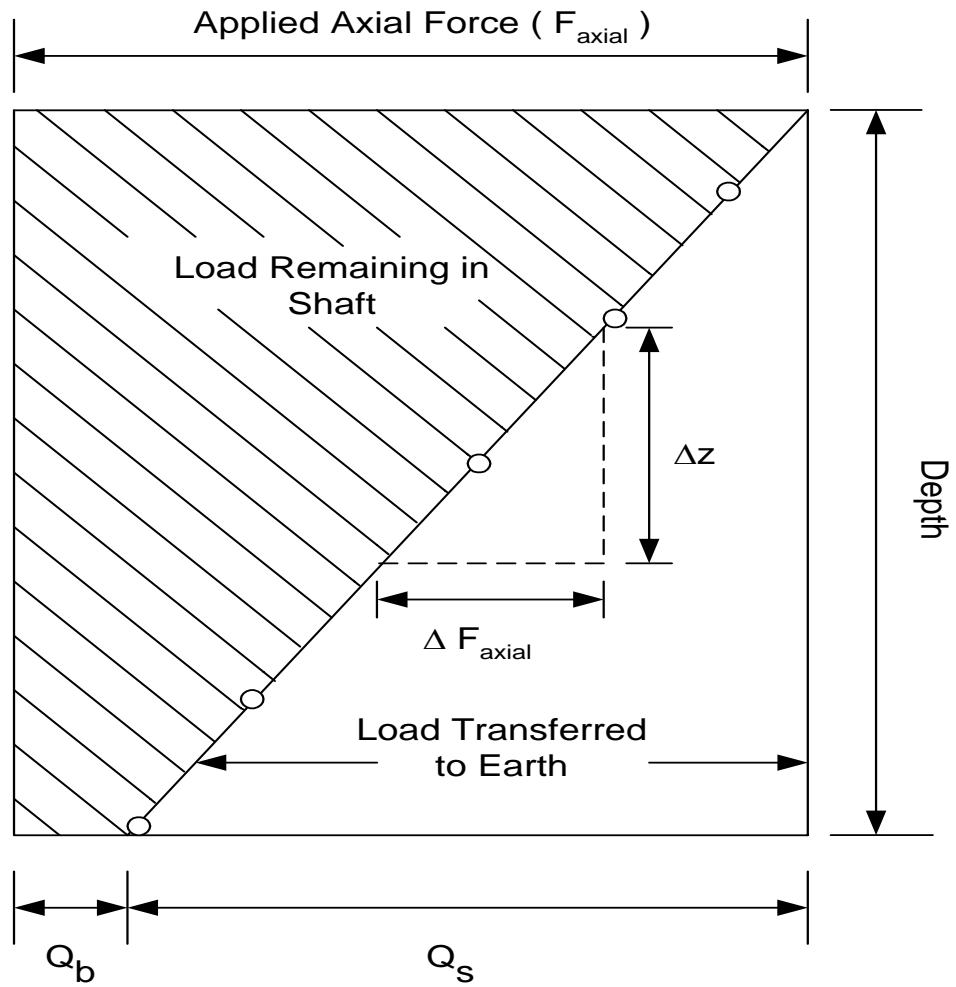
Figure 3.18- Distribution of load with depth for top-down load test (after Kyfor et al. 1992).
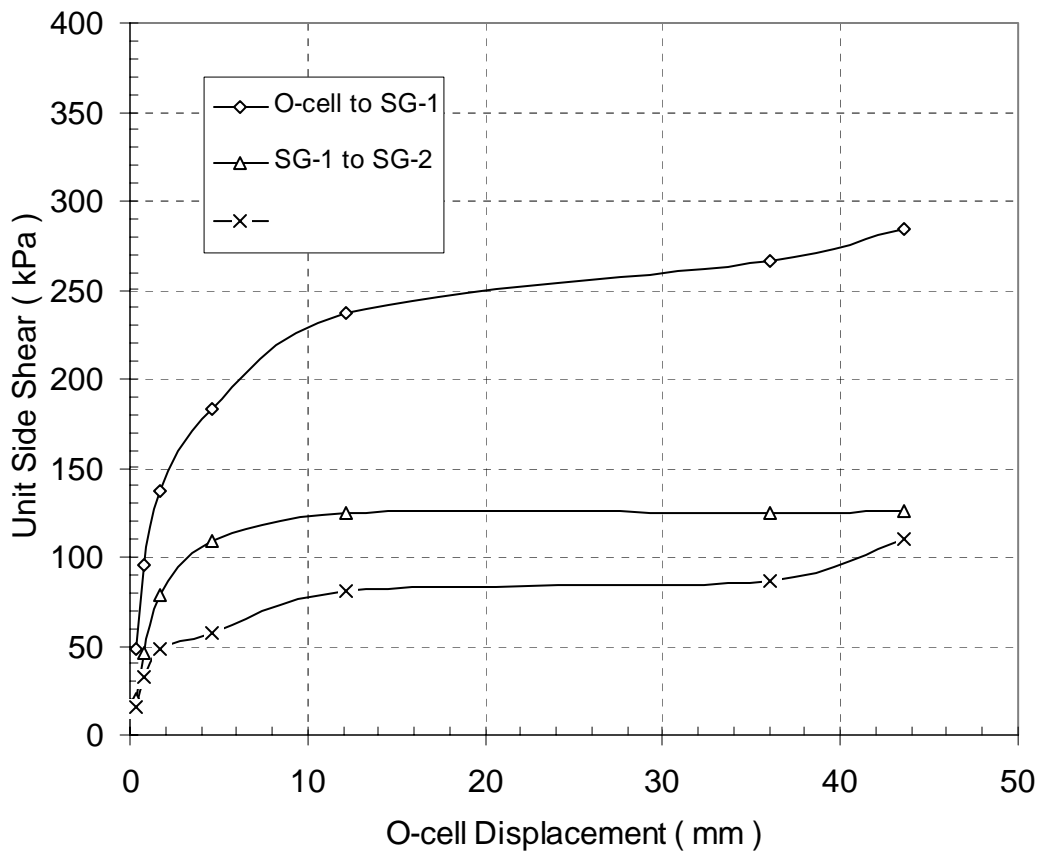
Figure 3.19- Unit side shear versus O-cell displacement relationships for several
segments of a drilled shaft.

Table 3.2- Comparison of the maximum mobilized unit side shear (after Shi
2002)

Depth Interval	Measured		FE Model		FE Model	
	O-cell^{TM} Test		O-cell^{TM} Test		Top Down Test	
	kPa	tsf	kPa	tsf	kPa	tsf
O-cell^{TM} to SG-1	746.9	7.8	536.3	5.6	344.7	3.6
SG-1 to SG-2	411.8	4.3	459.6	4.8	392.6	4.1
SG-2 to SG-3	363.9	3.8	392.6	4.1	402.2	4.2
SG-3 to SG-4	124.5	1.3	162.8	1.7	430.9	4.5

CHAPTER FOUR
LEXINGTON, MO. TEST SITE

A new bridge is proposed across the Missouri River in central Missouri. Foundation elements chosen for the piers in the river are to be drilled shafts socketed into bedrock. The bedrock at the bridge site consists of Pennsylvanian Age shales, siltstones, sandstones, limestones, and scattered coal beds. The unconfined compressive strength of this material varies from about 150 to 16,460 kPa (1.6 to 172 tsf). Present design methods used by MoDOT would dictate that axial load be carried in side shear only for drilled shafts socketed into bedrock. The ultimate side shear was calculated using methods developed by Horvath and Kenny (1979) or Reese and O'Neill (1988). These calculations showed that rock sockets with a diameter of 1.67 m (5.5 ft) would need to be as long as 18.3 m (60 ft) in order to carry the anticipated axial load. In order to develop a more economical design, it was decided to perform two Osterberg cell load tests at sites in the river close to the proposed bridge alignment. The Osterberg cell load tests where performed by Loadtest Inc. in May and June of 1999 and indicated that the bedrock had higher load capacities than estimated in the original design and that cost savings of around 1.8 million dollars could be realized.

A general geologic description of the Lexington test site is presented in this chapter followed by a summary of the engineering characteristics of the most pertinent strata. The construction and testing procedures for the two test shafts are then described, followed by presentation of the results from each load test. Because the bridge was designed, and load test results reported in SI units, all figures and tables are shown in SI units. Dual units are reported throughout the text.

4.1 Site Description

The Missouri Department of Transportation (MoDOT) is planning a realignment of Route 13 at Lexington, Missouri. The realignment includes a new bridge across the Missouri River, Section 19 and 22, Township 51 North, Range 26 West, about 3.9 km (2.4 miles) east or downstream of the present structure, which was opened to traffic in 1925.

The project is situated in the Missouri River alluvial plain in the central part of the state of Missouri as shown in Figure 4.1. The alluvial plain is mostly flat with some earthen levees constructed to protect row crop production. Currently, the river channel is located adjacent to the bluff on the southern limit of the plain. The Missouri River alluvial plain is about 1.1 kilometers (3350 feet) wide in the project area and the alluvial material consists of 1 to 2 meters (3.3 to 3.5 ft) of cohesive soil overlying sand with scattered gravel layers. The thickness of the alluvial materials in the floodplain North of the river varies from 26.5 to 33.3 meters (86.9 to 109.3 ft). Within the river, the alluvial sand and gravel layers range in thickness from 8.5 to 16 meters (27.9 to 52.5 ft) (HNTB 1998). The total length of the bridge is to be 1244.5 meters (4083 feet) with 25 spans of various lengths. The two main river spans will be 144 and 122 m (472 and 400 feet) in length as shown in Figure 4.2. The bridge will extend from the north abutment located in the flood plain southeast across the flood plain and river to the south abutment located on the bluff on the south side of the river.

Figure 4.1- Location sketch of Lexington bridge site.
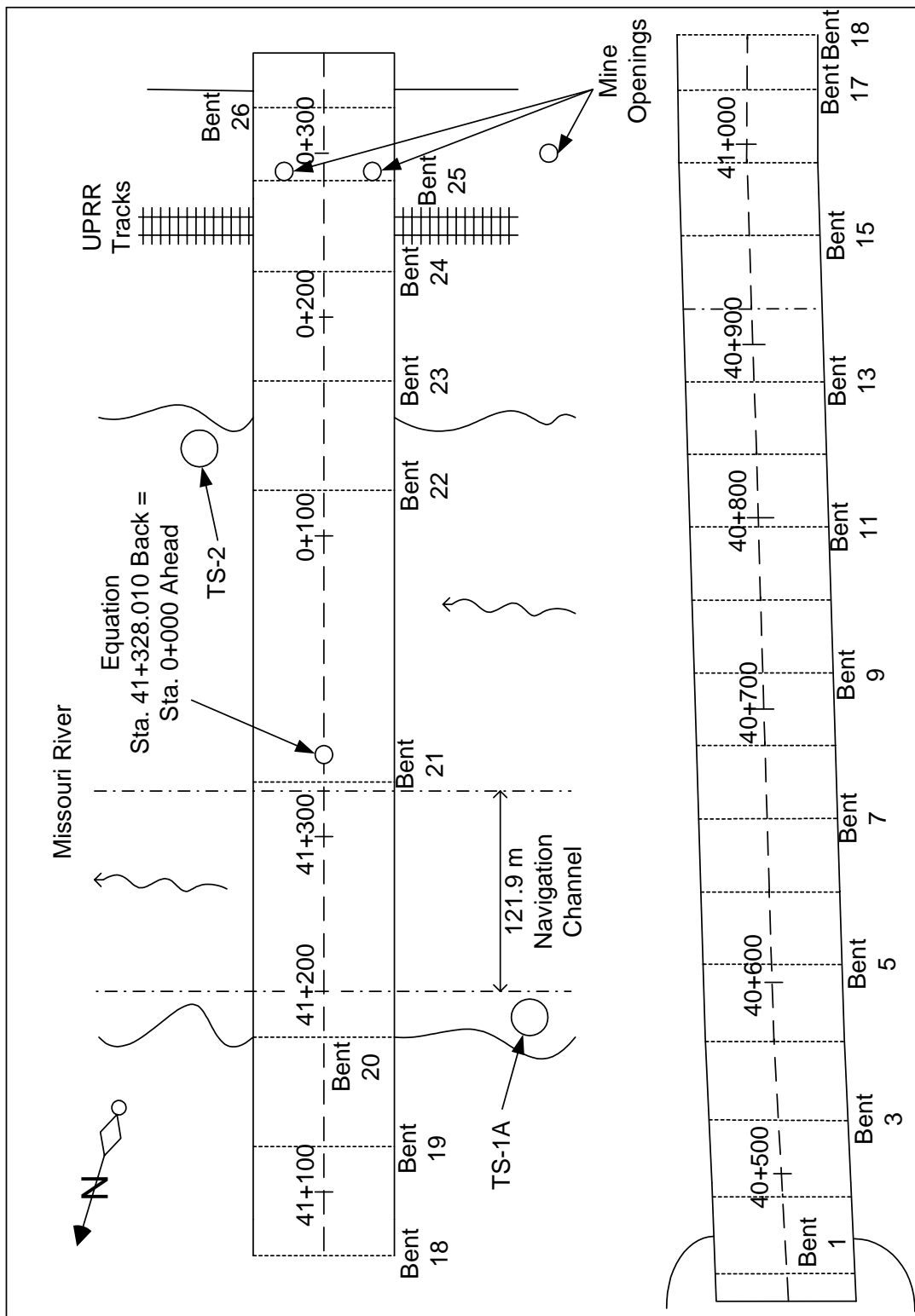
Figure 4.2- Plan view of Lexington bridge showing locations of bridge bents and test shafts

Foundation elements for Piers 1 through 18 were anticipated to be H-piles driven to bedrock. Drilled shafts socketed into the shale bedrock were anticipated for Piers 19 through 24, which included the three piers in the river (21, 22 and 23). Foundations for Piers 25 and 26 were anticipated to be H-piles driven in pre-bored holes in the shale bedrock. An elevation view of Piers 19 through 24 with anticipated rock sockets and test shafts TS-1A and TS-2 is provided in Figure 4.3. Piers 19, 23, and 24 will have 6 six shafts in a group; Piers 20 and 22 will have eight; and the mid-river pier, Pier 21, will have 15 shafts in a group. Rock sockets for Piers 19 through 21 will encounter from the base upwards the Croweburg Formation 4m (13.1 ft) thick, Verdigris Formation 3.4 m (11.2 ft) thick, and the Bevier Formation 10.6 m (34.8 ft) thick. Additionally, test shaft TS-1A would encounter the Fleming Formation below the Croweburg. Rock sockets at Pier 22 would encounter the Bevier Formation and the Lagonda Formation 10.8 m (35.4 ft) in thickness. Rock sockets for Piers 23 and 24 would encounter the Lagonda and the Mulky Formation 3.2 m (10.5 ft) in thickness. At Pier 25, the Higginsville Limestone was not continuous over the full footing and it was decided to prebore to about elevation 208.75 m and seat H-piles into the Little Osage Formation. At Pier 26, piles were anticipated to be pre-bored through the loess overburden and Altamont Formation and seated into the Bandera Formation. The geology of the site is described in the following paragraphs using the *Stratigraphic Succession in Missouri* (Thompson 1995) as a guide.
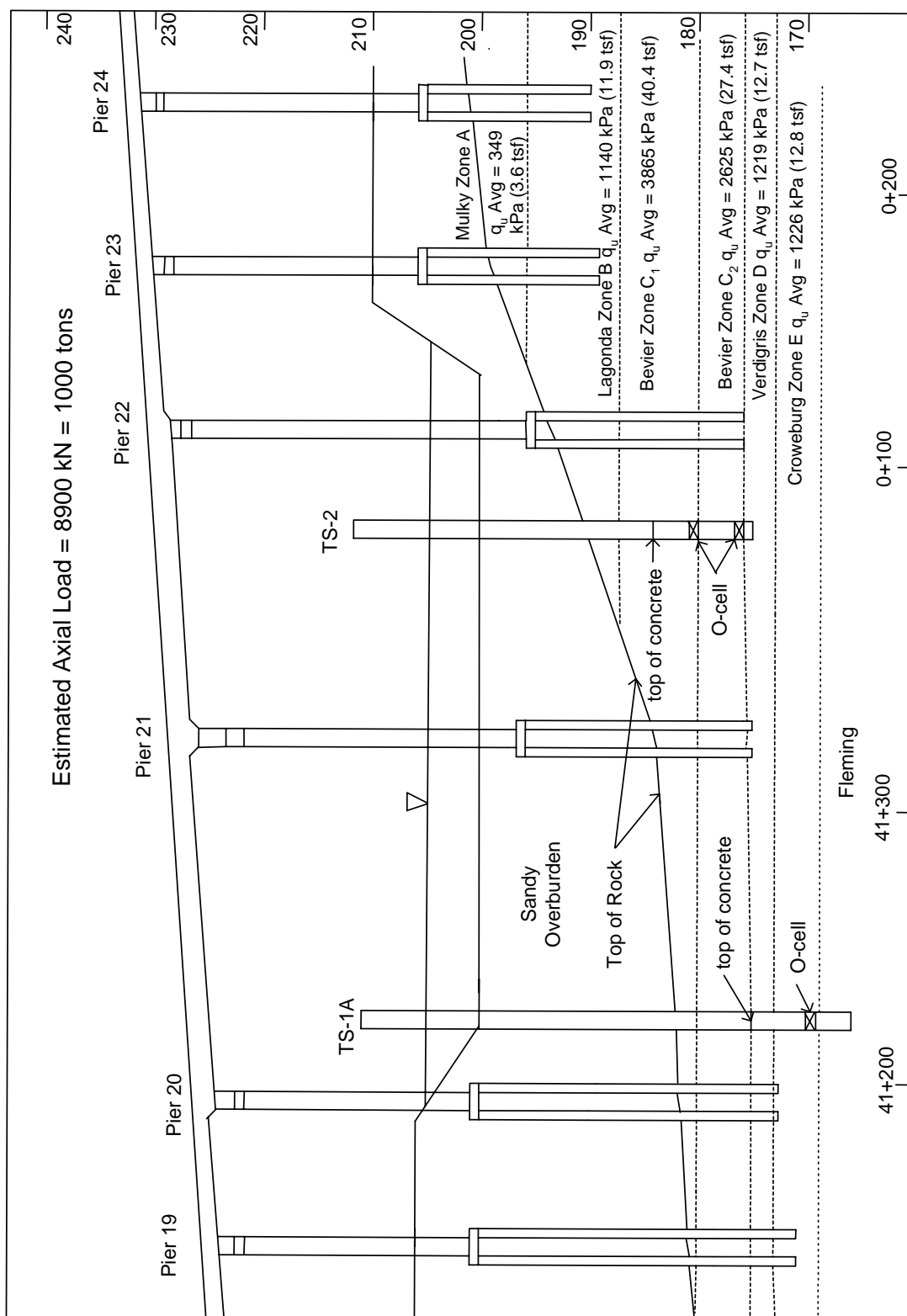
Figure 4.3- Elevation view of bridge showing piers 19 -24 and test shafts TS-1A and TS-2.

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4.2 Geology of the Area

The underlying bedrock at the test site is of lower Pennsylvanian Age, Desmoinesian Series, and is assigned to the Cherokee and Marmaton Groups. The Cherokee and Marmaton Groups are horizontally bedded and dip slightly in a northwesterly direction. The Cherokee Group contains most of the mineable coal beds in Missouri and is divided into the Krebs and Cabaniss Subgroups. Rock sockets for the drilled shafts are planned in the upper part of the Cabaniss Subgroup.

The Cabaniss Subgroup consists of sandstone, siltstone, underclay, limestone, and coal beds. Underclay is usually a very compact clay to claystone that underlies coal beds and commonly contains the roots of coal plants. The underclay may range from a few centimeters to several meters in thickness. The underclay is noted in MoDOT's drilling logs as a clay shale, poorly laminated. The subsurface investigation for the river piers encountered six of the eleven widely recognized successions of the Cabaniss Subgroup of the Cherokee Group. These are from the base upward: the Fleming Formation, the Croweburg Formation, the Verdigris Formation, the Bevier Formation, the Lagonda Formation, and the Mulky Formation. Each of these formations is described in the following paragraphs and the location of the rock sockets for the test shafts with respect to the formations is shown in Figure 4.4.
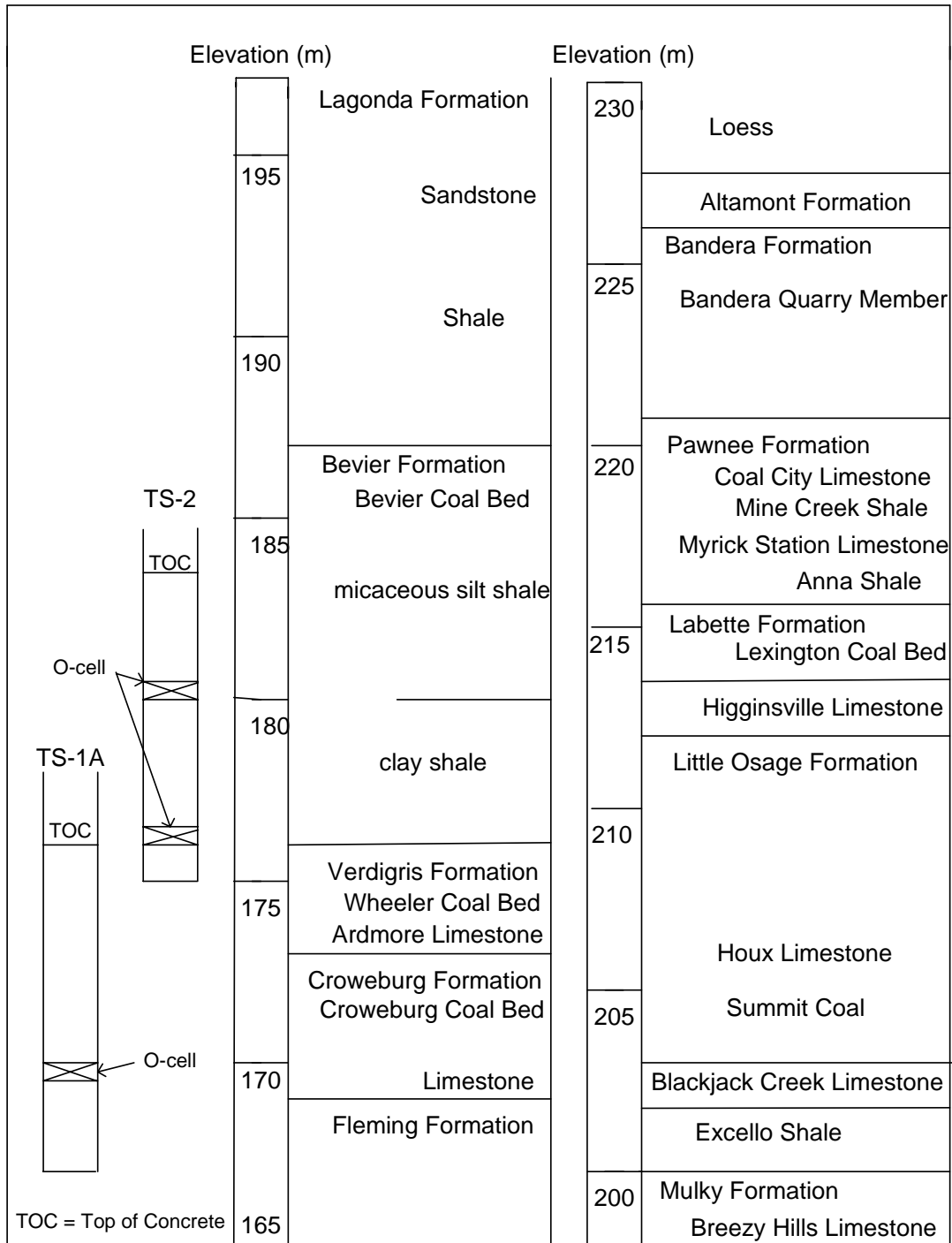
Figure 4.4- Stratigraphy of Lexington test site.

4.2.1 Fleming Formation

The Fleming Formation includes (from the base upward) a thin dark-gray fossiliferous limestone, a dark gray to black fissile shale, lenses of fine-grained sandstone and siltstone, an underclay, and the Fleming Coal Bed (Thompson 1995). This formation was encountered from about elevation 165.5 to 169.0 m and averages about 3.5 m (11.5 ft) in thickness. SPT blow counts in the Fleming Formation averaged 100 blows in 6 cm (2.4 in).

4.2.2 Croweburg Formation

The Croweburg Formation includes (from the base upward) a fossiliferous limestone, a gray-green calcareous clay shale, underclay, and the Croweburg Coal bed (Thompson 1995). This formation was encountered from about elevation 169 to 173 m and averages about 4 m (13.1 ft) in thickness. SPT blow counts in the Croweburg Formation averaged 100 blows in 8 cm (3.1 in). Liquid limits varied from 46 to 27 and the PI varied from 23 to 11. A jar slake test (Wood and Deo 1975 and Lutten 1977) performed on the gray clay shale indicated a jar slake index of 2 as shown in Figure 4.5.

Figure 4.5-   Jar slake index test results for Croweburg Formation: Elev. 172.27 m, slake index (2).

4.2.3 Verdigris Formation

The Verdigris Formation includes (from the base upward) gray-green clay shale to mudstone, a black fissile shale, gray thick bedded shaly limestone, (Ardmore Limestone Member), poorly laminated gray clay shale, (probably underclay), and the Wheeler coal bed (Thompson 1995). The Ardmore Limestone was encountered at about elevation 173.4 m. This formation was encountered from about elevation 173 to 176.4 m and averages about 3.4 m (11.2 ft) in total thickness. SPT blow counts in the Vedigris Formation averaged 100 blows in 12 cm (4.7 in). Liquid limits varied from 37 to 34 and the PI varied from 16 to NP. Jar slake tests performed on the gray clay shale varied from a jar slake index of 1 to 2 as shown in Figure 4.6.
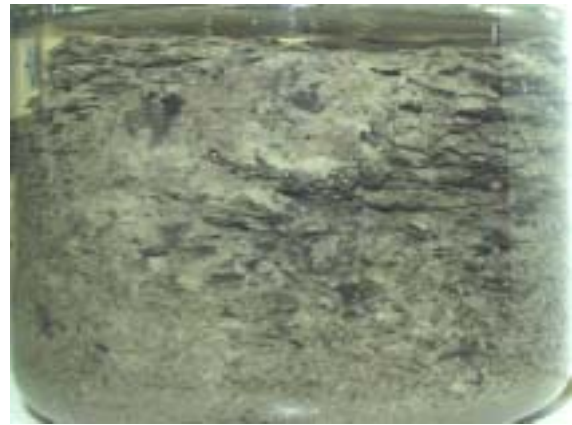
(a) Slake Index (1) (b) Slake Index (2)

Figure 4.6- Range of jar slake index test results for Verdigris Formation: (a) Elevation 176.75 m, slake index (1); (b) elevation 175.27 m, slake index (2).

4.2.4 Bevier Formation

The Bevier Formation includes (from the base upward) a gray clay shale, a micaceous silt shale to siltstone, a clay shale, and the Bevier coal bed (Thompson 1995). This formation was encountered from about elevation 176.4 to 187 m and averaged about

10.6 m (34.8 ft) in thickness. SPT blow counts in the Bevier Formation averaged 100

blows in 9 cm (3.5 in). Liquid limits varied from 25 in the upper Bevier (zone C1) to 39

in the lower Bevier (zone C2) and the PI varied from 2 to 16. Jar slake tests performed

on the dark gray to black shale varied from a jar slake index of 5 to 6 as shown in Figure

4.7.

(a) Slake Index (6) Bevier Zone C1   (b) Slake Index (5) Bevier Zone C2

Figure 4.7- Range of jar slake index test results for Bevier Formation: (a) Elevation
            183.95 m, slake index (6); (b) elevation 180.45 m, slake index (5).

4.2.5 Lagonda Formation

The Lagonda Formation includes a shale and siltstone or sandstone (Thompson

1995). This formation was encountered from about elevation 187 to 197.8 m and

averages about 10.8 m (35.4 ft) in thickness. SPT blow counts in the Lagonda Formation

averaged 100 blows in 13 cm (5.1 in).

4.2.6 Mulky Formation

The Mulky Formation includes an underclay, the Breezy Hills Limestone

Member, and the Mulky coal bed (Thompson 1995). The Breezy Hills Limestone was

encountered at about elevation 197.9 m. The formation was encountered from about

elevation 197.8 to 201.0 m and averages about 3.2 m (10.5 ft) in thickness.

4.3 Jar Slake Test of Shale Bedrock

The jar slake test is performed by immersing an oven dried sample of core in

water as described by Wood and Deo (1975) and Lutten (1977). The sample is observed

continuously for the first 10 minutes and carefully during the first 30 minutes. When a

reaction occurs, it happens primarily during this time frame. A final observation is made

after 24 hours. The condition of the piece is categorized as follows:

1. Degrades to a pile of flakes or mud

2. Breaks rapidly, forms many chips or both

3. Breaks slowly, forms many chips or both

4. Breaks rapidly, develops several fractures or both

5. Breaks slowly, develops few fractures or both

6. No Change

Jar slake index values for the shale formations at the Lexington site are summarized in

Table 4.1.

Table 4.1- Summary of jar slake index tests results for shale formations at Lexington site.

Formation	Zone	Elevation (m)	Jar Slake Index
Bevier	C1	183.95	6
Bevier	C1	181.15	6
Bevier	C2	180.45	5
Bevier	C2	178.57	5
Verdigris	D	177.77	1
Verdigris	D	176.75	1
Verdigris	D	175.27	2
Croweburg	D	172.75	2

4.4 Unconfined Compressive Strength of Bedrock at Piers 19 thru 24

Bedrock samples were taken with a standard split spoon sampler and a NX-sized

double-wall core barrel. The core was logged with the amount of core recovered and the

RQD being noted although MoDOT does not record RQD's in shale. Samples of the NX

core were returned to the laboratory for further testing. Unconfined compressive

strengths of the rock cores for Piers 19 through 24 varied from 140 kPa (1.5 tsf) to 25,830

kPa (269 tsf) with an overall average unconfined compressive strength of 2828 kPa (29.5

tsf). The bedrock profile was divided into six zones based on strata and trends in material

strengths, as shown in Table 4.2. The unconfined compressive strength of the rock core

varied considerably, influenced by the difficulty of obtaining representative samples of

the shale and the presence of scattered limestone layers. Unreasonably low values for the

unconfined compressive strength were not used to calculate the average strengths for the

layers. Higher values caused by scattered limestone layers were also discounted.

Table 4.2- Unconfined compressive strengths for rock cores at Lexington site.

Zone	Elevation	Formation	Avg. q_u		Range	Std. Dev.
	Meters		kPa	tsf	kPa	kPa
A	197.7-201	Mulky	225	2.3	110 - 340	117
B	187-197.7	Lagonda	1,570	16.4	140 - 7,520	1,775
C_1	180.4-187	Bevier	3,811	39.8	1020 - 8,105	2,210
C_2	176.4-180.4	Bevier	3,001	31.3	311 - 7,130	2,565
D	173.0-176.4	Verdigris	1,212	12.7	218 - 4,482	1,244
E	169-173.0	Croweburg	1,716	17.9	253 - 5,590	1,552
F	165.5-169	Fleming	544	5.7	150-1241	404

4.5 Foundation Design

Drilled shafts socketed into bedrock were chosen for the foundations of the piers immediately adjacent to the river and the river piers. Drilled shafts were chosen due to the thickness of the alluvium and the potential depth of scour. Scour is predicted to extend as much as 3 to 8 meters below the top of the shale bedrock.

Due to the alternating layers of shale, sandstone, siltstone, coal, and underclays, the rock socket design was based entirely on side resistance; potential resistance from end bearing was ignored. The ultimate unit side shear was determined by using Equations 2.11 and 2.18 presented in Chapter 2. The allowable unit side shear was determined by dividing the ultimate unit side shear by a safety factor of 2.5 as required by the American Association of State Highway and Transportation Officials (AASHTO 1996) manual on Standard Specifications for Highway Bridges. AASHTO allows the safety factor to be reduced to a value of 2.0 if a load test is performed. For a design load estimated to be about 8900 kN (1000 tons) and 1.68 meter (5.5 feet) diameter rock sockets, the required socket lengths would vary from 25.5 to 29 meters (83.6 to 95 feet). Rock sockets of this length would be very costly (HNTB 1996). The possibility of using larger diameter sockets, or increasing the number of sockets in a footing, was investigated but these alternatives were found to increase the footing size, which would in turn increase the potential scour depth and costs (HNTB 1996). In order to reduce the costs and allow for a practical design, it was decided to perform Osterberg cell load tests on two test shafts founded in various strata of the shale bedrock to better quantify available side shear resistance from the rock socket with the hope of producing more economical designs.

4.6 Construction of Test Shafts

Due to the difficulty with access to the sites by land and to avoid hindering river traffic, the two test shafts were located in the river but near the river banks. Test shaft one (TS-1) was planned to be located adjacent to the North river bank to evaluate side shear in Zones D, E, and F (Table 4.2) to provide design data for Piers 19 through 22. Test shaft two (TS-2) was to be constructed adjacent to the South river bank to evaluate side shear in Zones C_1, C_2, and D to provide design data for Piers 21 and 22, and possibly 23 and 24 (HNTB 1996). The location of the test shafts is shown in Figures 4.2 and 4.3. Test shaft one (TS-1) was located near the North bank of the river immediately west of Pier 20 and Test shaft two (TS-2) was located near the South bank immediately east of Pier 22 (Figure 4.2).

The two test shafts were constructed by Massman Construction using a Manitowoc 4100 series crane with drill assembly mounted on a barge as shown in Figure 4.8. Both shafts were permanently cased through the overburden soils with the permanent casing seated into the shale bedrock. The rock sockets were drilled using a bullet tooth rock auger. The drilling fluid used was water and concrete was poured using a tremie. The slump of the concrete was about 203 mm (8 in) when it left the concrete plant and about 102 mm (4 in) when it was pumped into the shafts. The concrete mix included a four-hour retarder. Loadtest Inc. provided the Osterberg cells, instrumentation, and gages and performed the load tests. Both test shafts were impacted by high river levels and scheduling conflicts. As described in more detail below, test shaft TS-1 had to be abandoned due to caving of the shale bedrock, and was subsequently replaced by another test shaft denoted as TS-1A.

Figure 4.8- Manitowoc 4100 series crane with drill assembly.

4.6.1 Construction of Test Shafts TS-1 and TS-1A

The contractor mobilized to the site and began driving sheet piling for a work platform and ice deflector on February 15, 1999. The overburden soils for shaft TS-1 were excavated and 31 m (102 ft) of casing was set into the shale bedrock on March 30, 1999. The excavation of the rock socket was completed on April 13, 1999. The river subsequently rose above the construction platform and inundated the shaft. The river subsided below the work platform on April 19, 1999. The O-cell^{TM} was removed and the rock socket was over-reamed on April 22nd in an attempt to freshen the sidewalls of the socket. The river then rose above the platform on April 23rd for a second time. Work resumed on May 12, 1999 when an additional sonar caliper test of the rock socket was performed. Sonar results indicated that the socket had caved to more than double the

original diameter at some depths. TS-1 was therefore abandoned and a new test shaft, TS-1A, was drilled 15 meters upstream of TS-1. The collapse of test shaft TS-1 has led to a specification change that requires rock sockets to be excavated and the shaft concrete to be placed within 3 days for shafts constructed in shales. Further specification changes require the use of polymer slurry in drilled shafts constructed in shale that cannot be constructed "in the dry."

The rock socket for test shaft TS-1A was drilled on June 15th through 16th, 1999. Figure 4.9 shows a schematic of TS-1A. The shaft was calipered on June 17, 1999 using a sonar caliper as shown in Figure 4.10. The carrying frame with one 660 mm (26 in) O-cell[TM] and various instrumentation was then placed into the rock socket as shown in Figure 4.11. The carrying frame was constructed from C-4 channel section as shown in Figure 4.12. The shaft concrete was also placed into the rock socket on June 17, 1999. A sonar caliper log for TS-1A is presented in Figure 4.13   Loadtest personnel arrived on the site on June 22, 1999 and started the test at 10:35 am.   The test was completed at 2:00 pm.
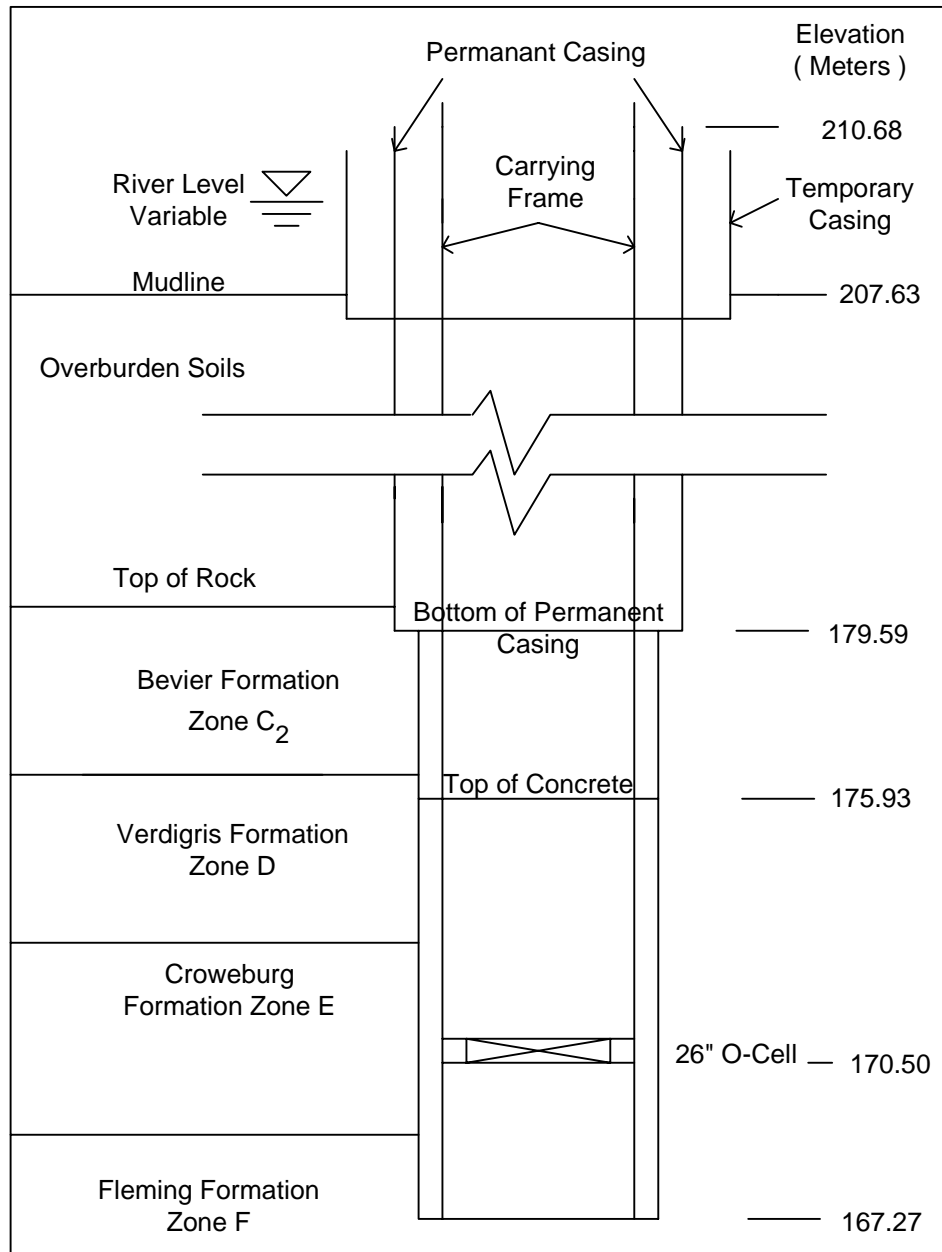
Figure 4.9- Schematic of test shaft TS-1A.

Figure 4.10- Sonar caliper prior to placement in test shaft excavation.

Figure 4.11- Preparing to lower carrying frame and O-cell^{TM} into test shaft TS-1A.

Figure 4.12- Carrying frame and instrumentation for test shaft TS-1A.

112	Feet	- 43.9"	Diameter
112.6	Feet	- 43.8"	Diameter
113	Feet	+ 44.4"	Diameter
113.6	Feet	- 44.52"	Diameter
114	Feet	- 44.52"	Diameter
114.6	Feet	- 43.92"	Diameter
115	Feet	- 44.04"	Diameter
115.5	Feet	- 43.56"	Diameter
116	Feet	- 44.04"	Diameter
116.5	Feet	- 44.28"	Diameter
117	Feet	- 43.68"	Diameter
117.5	Feet	- 43.68"	Diameter
118	Feet	- 44.52"	Diameter
118.5	Feet	- 44.4"	Diameter
119	Feet	- 44.86"	Diameter
119.5	Feet	- 43.8"	Diameter
120	Foot	- 44.16"	Diameter
120.5	Feet	- 44.04"	Diameter
121	Feet	+ 43.9"	Diameter
121.5	Feet	+ 44.40"	Diameter
122	Feet	- 43.8"	Diameter
122.5	Feet	- 44.4"	Diameter
123	Feet	- 43.56"	Diameter
123.5	Feet	+ 43.44"	Diameter
124	Feet	+ 43.32"	Diameter
124.5	Feet	- 43.32"	Diameter
125	Feet	- 43.44"	Diameter
125.5	Feet	- 43.06"	Diameter
126	Feet	- 43.32"	Diameter
126.5	Feet	+ 43.68"	Diameter
127	Feet	+ 43.32"	Diameter
127.5	Feet	- 43.44"	Diameter
128	Feet	- 42.96"	Diameter
128.5	Feet	- 43.08"	Diameter
129	Feet	- 43.80"	Diameter
129.5	Feet	- 43.56"	Diameter
130	Feet	- 44.28"	Diameter
130.5	Feet	- 42.84"	Diameter
131	Feet	- 43.44"	Diameter
131.5	Feet	- 43.32"	Diameter
132	Feet	- 42.96"	Diameter
132.5	Feet	- 42.96"	Diameter
133	Feet	- 43.20"	Diameter
133.5	Feet	- 43.08"	Diameter
134	Feet	- 44.52"	Diameter
134.5	Feet	- 44.64"	Diameter
135	Feet	- 45.84"	Diameter
135.5	Feet	- 47.04"	Diameter
136	Feet	- 48.24"	Diameter
136.5	Feet	- 44.16"	Diameter
137	Feet	- 42.96"	Diameter
137.5	Feet	- 43.56"	Diameter
138	Feet	- 43.08"	Diameter
138.5	Feet	- 42.96"	Diameter
139	Feet	- 43.20"	Diameter
139.5	Feet	- 42.72"	Diameter
140	Feet	- 42.48"	Diameter
140.5	Feet	+ 42.24"	Diameter
141	Feet	+ 42.60"	Diameter
141.5	Feet	- 42.60"	Diameter

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Left View

Right View

Figure 4.13- Sonar caliper log of rock socket for test shaft TS-1A.

4.6.2 Construction of Test Shaft TS-2

Test shaft TS-2 included two Osterberg load cells to allow evaluation of unit side shear in separate strata. Figure 4.14 shows a schematic of test shaft TS-2. The contractor mobilized and began driving sheet piling for the work platform and ice deflector on February 18, 1999. Temporary casing was set on April 1, 1999. The overburden soils were excavated and 17.1 m (56 ft) of permanent casing was set into the shale bedrock on April 2nd. The rock socket was drilled to about 1.5 m (5ft) above the planned top of the rock socket concrete (Elev. 184.4 m) on April 6, 1999. Completion of the shaft was then delayed due to scheduling conflicts. On April 14, 1999 the socket was drilled to about 1 meter below the location for the upper Osterberg cell (Elev. 180.17 m). During the following days, the river level rose above the construction platform and likely topped the permanent casing. When the river level dropped below the platform on April 21, 1999, TS-2 was drilled to the planned depth and cleaned by airlifting. The socket was calipered with a sonar unit and the carrying frame with two O-cells^{TM} was set on April 22nd as shown in Figures 4.15 and 4.16. Sonar caliper logs for TS-2 are presented in Figures 4.17 and 4.18.

Since the test shafts were located in the river, the delivery of concrete to the site involved a 2.5 kilometer (1.5 mile) barge trip. Delays occurred in placing concrete in TS-2 due to clogging of the tremie pipe and, by the end of the day on April 22nd, only the bottom cell was encased in concrete. This caused a cold joint at an elevation of approximately 178.5 meters. The permanent casing was capped as the river rose above the platform on April 23rd. On April 26th air and water were pumped to the cold joint to

attempt to suspend the dirt and debris that had accumulated and a second attempt to complete the pour was made. The slump of the concrete was about 102 mm (4 in). Problems with the concrete pump and tremie again delayed the pour. The cold joint was again flushed on April 27th and the concrete pour was completed. Loadtest personnel arrived on the site and completed the test on May 3, 1999.

Figure 4.14- Schematic of test shaft TS-2.

Figure 4.15- Carrying frame with two Osterberg load cells for test shaft TS-2.

Figure 4.16- Lowering carrying frame and O-cells for test shaft TS-2.

42.84 Diameter = 81 Feet
43.08 Diameter = 81.5 Feet
42.72 Diameter = 82 Feet
43.20 Diameter = 82.5 Feet
43.20 Diameter = 83 Feet
42.96 Diameter = 83.5 Feet

43.20 Diameter = 84 Feet
43.20 Diameter = 84.5 Feet
43.20 Diameter = 85 Feet
43.20 Diameter = 85.5 Feet
43.32 Diameter = 86 Feet
43.44 Diameter = 86.5 Feet
43.20 Diameter = 87 Feet
43.20 Diameter = 87.5 Feet

43.20 Diameter = 88 Feet
43.20 Diameter = 88.5 Feet
43.20 Diameter = 89 Feet

43.44 Diameter = 90 Feet
43.68 Diameter = 90.5 Feet
43.44 Diameter = 91 Feet
43.44 Diameter = 91.5 Feet
47.28 Diameter = 92 Feet

47.52 Diameter = 92.5 Feet
47.88 Diameter = 93 Feet

49.20 Diameter = 93.5 Feet
49.68 Diameter = 94 Feet
50.40 Diameter = 94.5 Feet
48.72 Diameter = 95 Feet
47.04 Diameter = 95.5 Feet
46.08 Diameter = 96 Feet

46.44 Diameter = 96.5 Feet

45.84 Diameter = 97 Feet
46.56 Diameter = 97.5 Feet

46.80 Diameter = 98 Feet

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Left View   Right View

Figure 4.17- Sonar caliper log of rock socket for test shaft TS-2 (81-98 ft from top of casing).

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49.44  Diameter  -   98.5 Feet
49.68  Diameter  -   99 Feet
50.16  Diameter  -   99.5 Feet

48.00  Diameter  -   100 Feet
49.32  Diameter  -   100.5 Feet
44.64  Diameter  -   101 Feet
44.16  Diameter  -   101.5 Feet
44.76  Diameter  -   102 Feet
44.64  Diameter  -   102.5 Feet

45.12  Diameter  -   103 Feet

45.48  Diameter  -   103.5 Feet
46.44  Diameter  -   104 Feet

45.96  Diameter  -   104.5 Feet
45.72  Diameter  -   105 Feet
45.60  Diameter  -   105.5 Feet

45.84  Diameter  -   106 Feet
46.20  Diameter  -   106.5 Feet
47.40  Diameter  -   107 Feet

48.84  Diameter  -   107.5 Feet

48.24  Diameter  -   108 Feet

49.80  Diameter  -   108.5 Feet
48.96  Diameter  -   109 Feet
47.64  Diameter  -   109.5 Feet
46.32  Diameter  -   110 Feet
45.48  Diameter  -   110.5 Feet
45.72  Diameter  -   111 Feet
45.24  Diameter  -   111.5 Feet
44.40  Diameter  -   112 Feet
43.80  Diameter  -   112.5 Feet

44.40  Diameter  -   113 Feet
44.04  Diameter  -   113.5 Feet
43.92  Diameter  -   114 Feet
44.40  Diameter  -   114.5 Feet
43.80  Diameter  -   115 Feet
44.40  Diameter  -   115.5 Feet
44.16  Diameter  -   116 Feet
44.88  Diameter  -   116.5 Feet

43.68  Diameter  -   117 Feet
39.48  Diameter  -   117.5 Feet
```

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Left View   Right View

Figure 4.18- Sonar caliper log of rock socket for test shaft TS-2 (98.5-117.5 ft from top
of   casing).

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4.7 Load Test Setup and Procedures

Load tests for test shafts TS-1A and TS-2 were performed by Loadtest Inc. on June 22, 1999 and May 3, 1999, respectively. The test setup and instrumentation for each test are described in the following sections along with the loading procedure utilized for each test.

4.7.1 Test Shaft TS-1A Setup and Procedure

The 660 mm (26 in) diameter O-cell^{TM}, with its base located 3.23 m (10.6 ft) above the tip of the rock socket was pressurized to assess the combined end bearing and side shear below the O-cell^{TM} and side shear above the cell. The O-cell^{TM} was pressurized in 36 equal increments of 0.48 MN (54.3 tons) to a maximum load of 17.39 MN (1955 tons). The loading increments are denoted as 1L-1, 1L-2, 1L-3, etc. and the unloading events are denoted as 1U-1, 1U-2, 1U-3, etc. Although the capacity of the cell was not reached, the capacity of the available pressure gage was reached at a load of 17.39 MN (1955 tons) and the test had to be stopped. The O-cell^{TM} was then unloaded in 7 equal increments and the test was concluded. Other than the problem with the gage capacity no significant problems were encountered in performing the test.

Expansion of the O-cell^{TM} was measured by three LVWDTs positioned between the lower and upper plates of the O-cell^{TM}. Test shaft TS-1A instrumentation is shown in Figure 4.19 and a summary of test shaft dimensions is given in Table A.1 in Appendix A. Compression of the shaft above the O-Cell^{TM} was measured by a pair of embedded compression telltales (ECTs). Two digital dial gages attached to a reference beam monitored the top of shaft (carrying frame) movement. Four levels of three sister bar

vibrating wire strain gages were installed in the shaft: one level below the cell and three above. The strain gages were used to assess load transfer in the shaft.
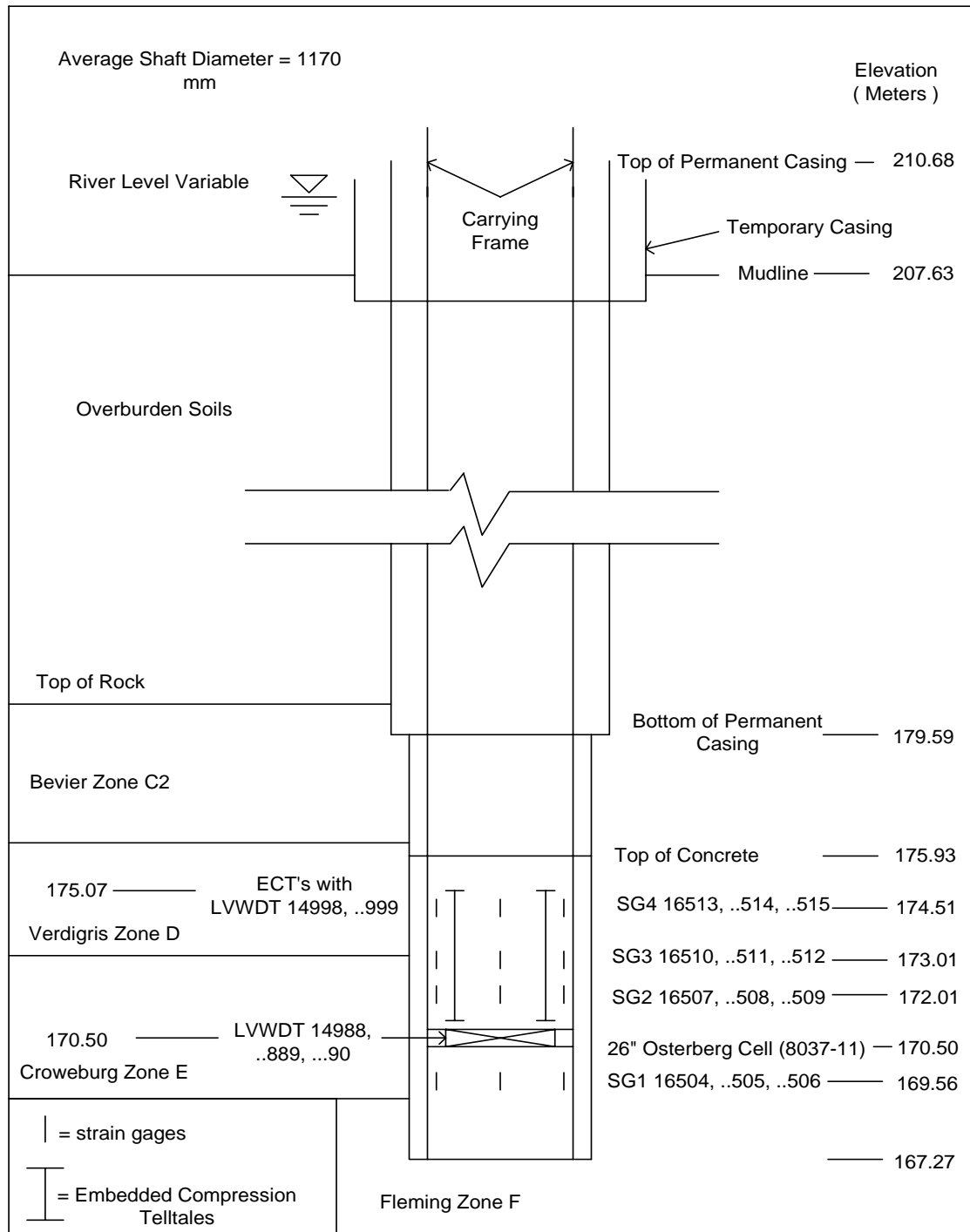
Figure 4.19- Schematic of test shaft TS-1A showing location of instrumentation.

4.7.2 Test Shaft TS-2 Setup and Procedure

The load test for test shaft TS-2 was performed in three stages to evaluate load transfer in different segments of the shaft. The three stages are as follows:

Stage 1: In Stage 1, the lower 660 mm (26 in) diameter O-Cell^{TM}, with its base located 1.52 m (5 ft) above the tip of the rock socket, was pressurized to assess the combined end bearing and side shear below the lower O-Cell^{TM} using the side shear above the lower and upper cells as the reaction. The lower O-Cell^{TM} was pressurized in 13 even increments of 0.8 MN (90 tons) to a maximum load of 10.59 MN (90 tons), at which point the combined end bearing and side shear below the cell were approaching ultimate capacity. The lower O-Cell^{TM} was then unloaded in 5 increments and Stage 1 was concluded.

Stage 2: In Stage 2, the hydraulic line to the lower O-cell^{TM} was left open to allow compression of the cell. The upper 660 mm (26 in) diameter O-cell^{TM} was then pressurized to assess side shear of the shaft between the upper and lower O-cells^{TM} using the side shear above the upper O-cell^{TM} as the reaction. By allowing the lower O-Cell^{TM} to drain, no end bearing resistance was provided below the lower O-cell^{TM}. The upper O-cell^{TM} was pressurized in 13 even increments of 0.8 MN (90 tons) to a maximum load of 10.59 MN (1190 tons). The upper O-cell^{TM} was not unloaded.

Stage 3: In Stage 3, the hydraulic line to the lower cell was closed (therefore providing end bearing resistance) before the next loading increment was applied to the upper O-cell^{TM}. The combined end bearing and side shear below the upper O-cell^{TM} where then used to assess the side shear above the upper O-cell^{TM}. The upper O-cell^{TM} was loaded in four additional increments of 0.8 MN (90 tons) to a maximum load of

13.35 MN (1500 tons), at which point the upper O-cell^{TM} reached its maximum stroke and depressurized.

Expansion of the O-Cells^{TM} was measured by three LVWDTs positioned between the lower and upper plates of the O-Cells^{TM}. Test shaft TS-2 instrumentation is shown in Figure 4.20 and a summary of test shaft dimensions is given in Table A.5 in Appendix A. Compression of the shaft between the two levels of O-Cells^{TM} was measured by a pair of embedded compression telltales (ECTs). Compression of the shaft above the upper O-cell^{TM} was also measured by a pair of ECTs. Two digital dial gages attached to a reference beam monitored the top of shaft (carrying frame) movement. Four levels of three sister bar vibrating wire strain were installed in the shaft, two levels above the lower O-Cell^{TM} and two levels above the upper O-cell^{TM}. The strain gages were used to assess load transfer in the shaft.

4.8 General Test Results

The results of the Osterberg cell load tests are presented in the following sections for test shafts TS-1A and TS-2. Detailed load test data is presented in Appendix A. Load- displacement plots are presented from which an "equivalent" top-down load-displacement curve is constructed. Creep is plotted versus the O-cell^{TM} load to determine the creep limit. The distribution of axial force with depth or elevation for various load increments, generated from the strain gage data, is then presented along with unit side shear values calculated for various segments of the shaft. Plots of unit side shear versus O-cell^{TM} movement are used to determine if the maximum unit side shear was achieved.
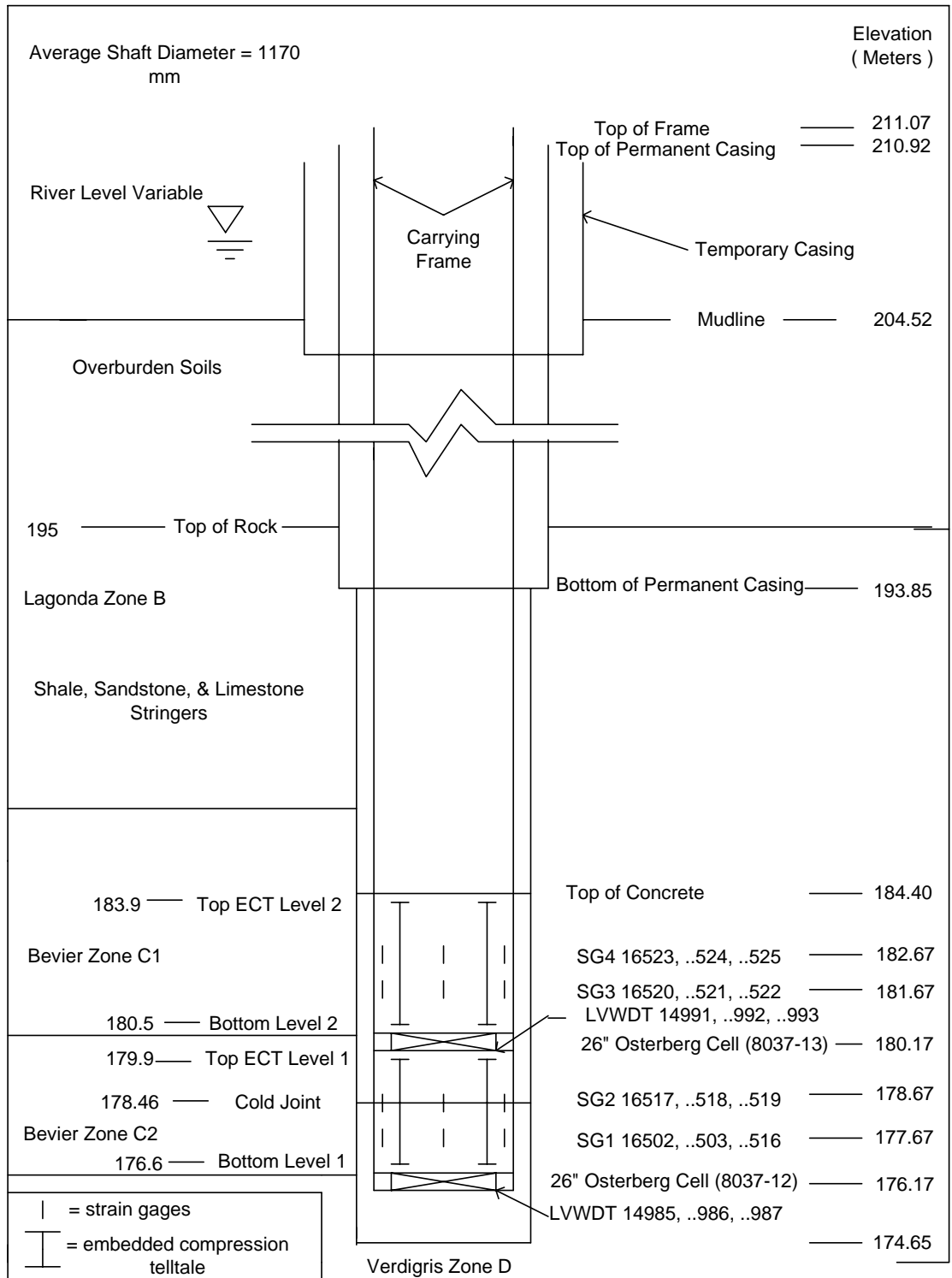
Figure 4.20- Schematic of test shaft TS-2 showing location of instrumentation.

4.8.1 Test Results for TS-1A

The measured load-displacement response for the load test on test shaft TS-1A is shown in Figure 4.21. The maximum applied load occurred at the 36[th] increment of load for the first and only load interval (1L-36). The maximum load was equal to 17.5 MN minus the buoyant weight of the shaft above the O-Cell[TM] (0.11 MN), for a net applied load of 17.39 MN (1955 tons). At this load, the O-Cell[TM] had expanded 19.1 mm (0.75 in) with 5.5 mm (0.22 in) of upward displacement and 13.6 mm (0.54 in) of downward displacement. The ultimate capacity was not reached in the rock socket either above or below the O-Cell[TM] although the downward loading appears nearer to failure than the upward loading.
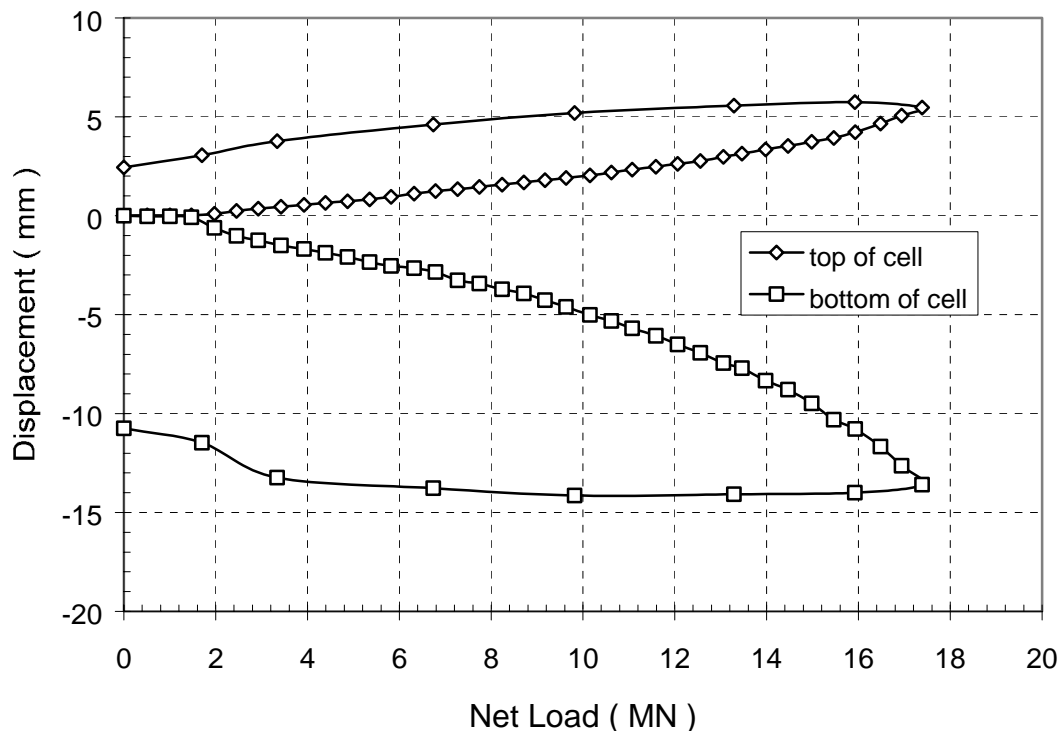
Figure 4.21- Measured load-displacement curves for downward and upward loading of test shaft TS-1A.

Equivalent top-down load-displacement curves were determined as described in Chapter 3. The equivalent top-down load-displacement curves for shaft TS-1A are shown in Figure 4.22. One of the curves shown was computed assuming a rigid shaft, while the other accounts for additional elastic compression that would occur in a top-down test as described in Chapter 3. The curve is extended out to a displacement of 13.6 mm (0.54 in) by extrapolating the O-Cell^{TM} data for the upper portion of the shaft as described in Chapter 3.
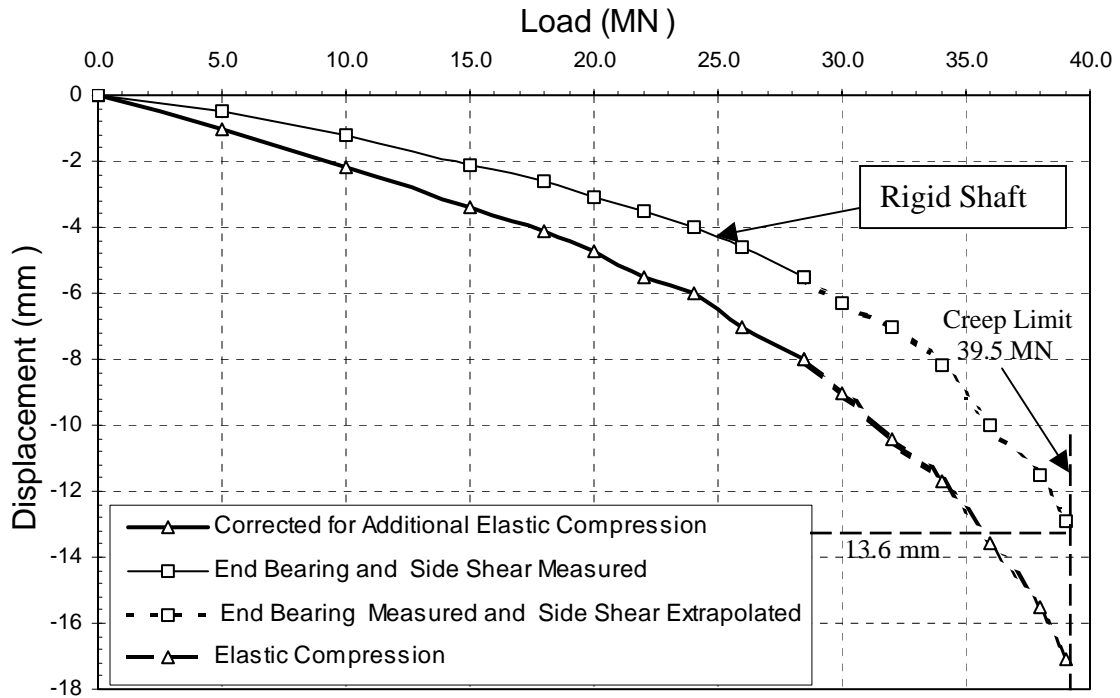
Figure 4.22- Equivalent top-down load-displacement curve for test shaft TS-1A.

The distribution of axial force with depth or elevation at various load increments is generated from strain gage data, the equivalent modulus of the shaft, and the cross-sectional area of the shaft as described in Chapter 3 (Eq. 3.7). The distribution of axial force determined for several load increments applied to TS-1A is shown in Figure 4.23. The strain gage data is provided in Appendix A. On the day of the test, the compressive

strength of test cylinders made from the shaft concrete was 28.1 MPa (4075 psi). This

combined with the area of reinforcing steel and shaft diameter, was used to determine an

average shaft stiffness (AE) of 25,506 MN (5,730,000 kips).
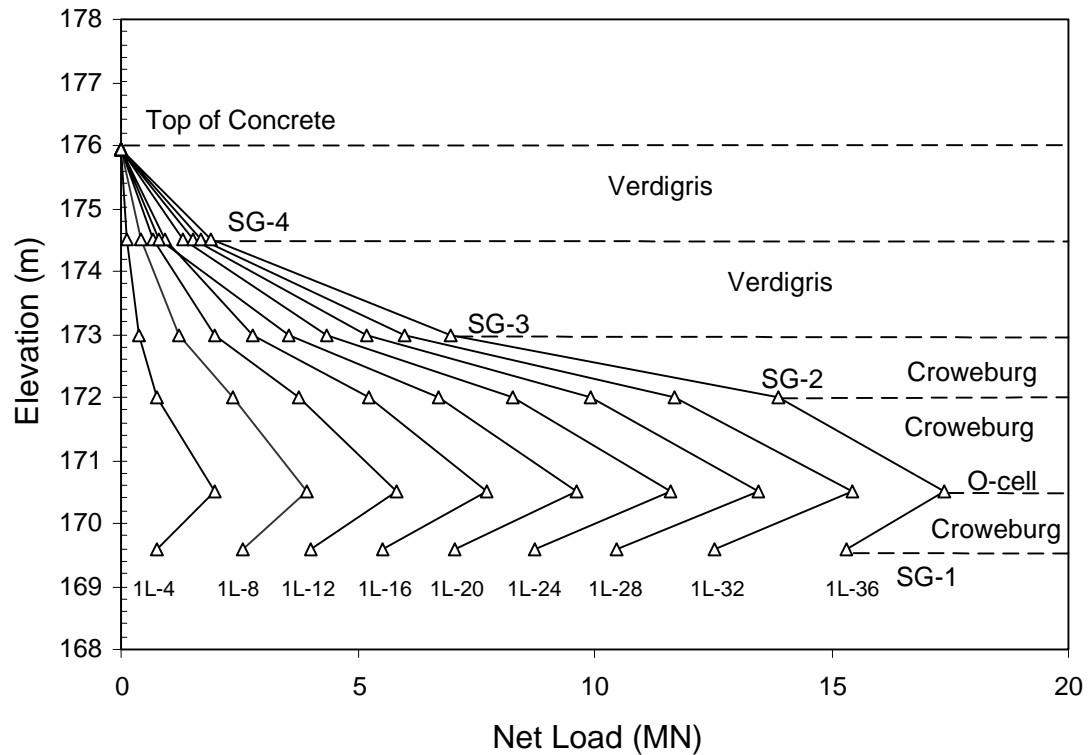
Figure 4.23- Distribution of axial force for test shaft TS-1A.

The magnitude of the unit side shear mobilized for a segment of the shaft is

calculated as the change in axial force, ΔF_i, over the length of the segment divided by

the surface area of the shaft segment. The average unit side shear above the O-Cell[TM]

was found to be 873 kPa (9.1 tsf). Average unit side shear mobilized in the socket below

the O-Cell[TM] cannot be estimated directly since an unknown amount of end bearing

resisted the load. If the unit side shear value for the segment of shaft below the O-Cell[TM]

is assumed to be equal to the unit side shear between the O-cell[TM] and the level 1 strain

gages (SG-1), the unit end bearing value is computed to be 9622 kPa (100.5 tsf).

The mobilized unit side shear for each load increment is plotted versus O-cell^{TM} movement in Figure 4.24. The mobilized unit side shear curves indicate that the side shear has been fully mobilized in the shaft segments immediately adjacent to the O-cell^{TM} (between the O-cell^{TM} and SG-1 and the O-cell^{TM} and SG-2). The mobilized unit side shear curves for the other segments are still increasing, which indicates that the unit side shear values determined for these strata are less than ultimate values. The maximum values of unit side shear determined for each strata are shown in Table 4.3 and plotted in Figure 4.25. The overall maximum value of unit side occurred in the segment of the socket between the level 2 (SG-2) and level 3 (SG-3) strain gages in the Croweburg Formation and was equal to 1983 kPa (20.7 tsf). However, the unit side shear was not the ultimate value (Figure 4.24).

The creep limit is determined by plotting the displacement that occurs over the time interval 2 to 4 minutes after application of a load while the load is maintained constant as shown in Figures 4.26 and 4.27. As shown in the figures, a creep limit was not reached for either the upper side shear or the combined lower side shear and end bearing. A maximum displacement of 13.6 mm (0.54 in) occurred in the lower segment of the shaft during the O-cell^{TM} load test. This data indicates that since a creep limit was not determined, significant creep would not occur for a top loaded shaft until a displacement greater than 13.6 mm (0.54 in) is exceeded by some unknown amount. The displacement of 13.6 mm (0.54 in) correlates to a load of 39.5 MN (4440 tons) on the rigid top-down load-displacement curve as shown in Figure 4.22.
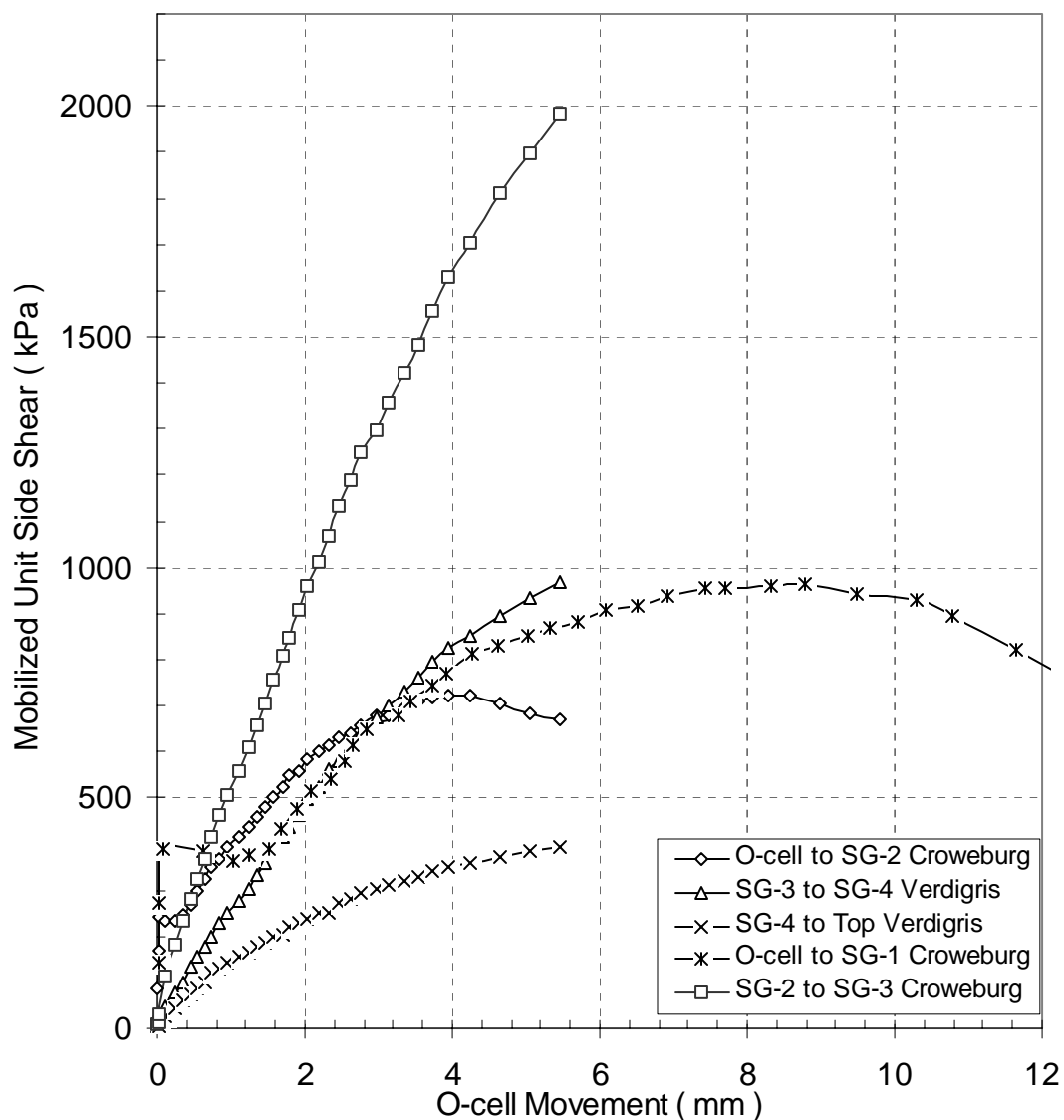
Figure 4.24- Mobilized unit side shear versus O-cell^{TM} movement for test shaft TS-1A.

Table 4.3- Unit side shear values calculated from strain gage data for test shaft TS-1A.

Load Transfer Zone	Strata	Elevation (m)	Unit Side Shear	
		From – To	kPa	tsf
SG-4 to Top of Shaft	Verdigris	174.51 – 175.93	391	4.1
SG-3 to SG-4	Verdigris	173.01 – 174.51	968	10.1
SG-2 to SG-3	Croweburg	172.01 – 173.01	1983	20.7
O-Cell^{TM} to SG-2	Croweburg	170.5 – 172.01	723	7.6
SG-1 to O-Cell^{TM}	Croweburg	169.56 – 170.5	963	10.1
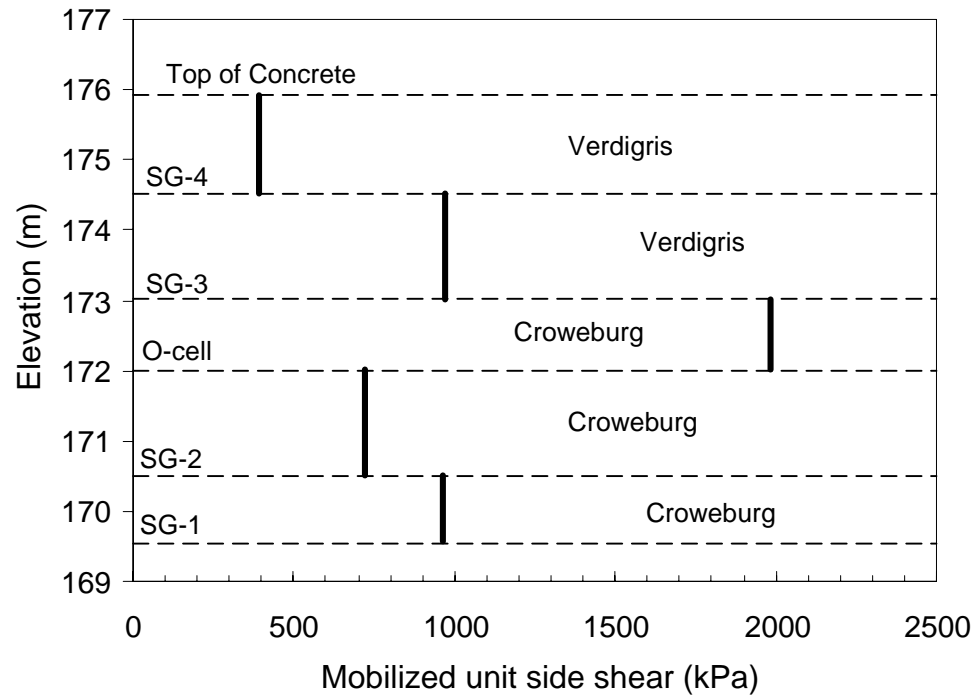
Figure 4.25- Mobilized unit side shear values calculated from strain gage data for test shaft TS-1A.
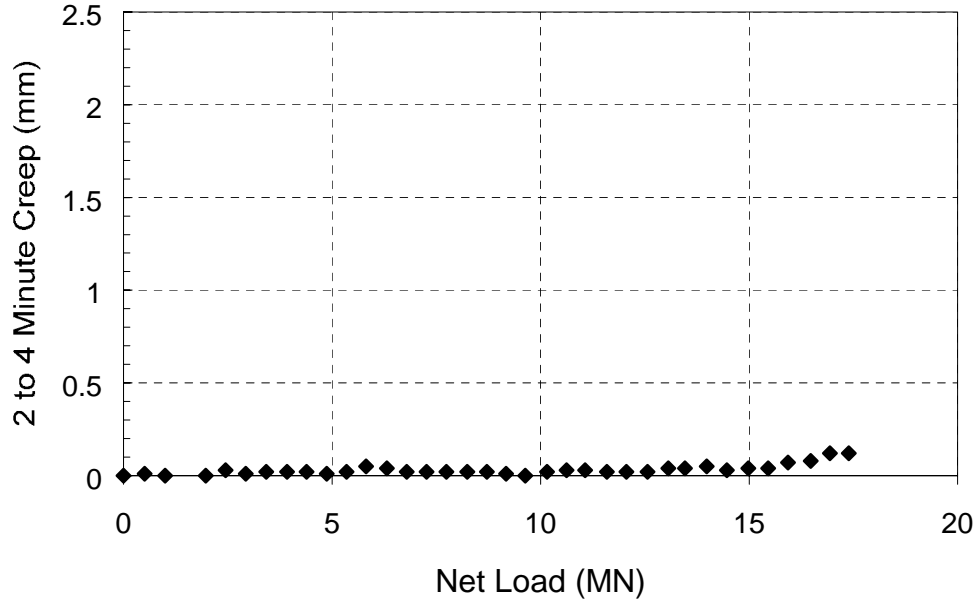
Figure 4.26- Creep displacement for the upper portion of test shaft TS-1A.
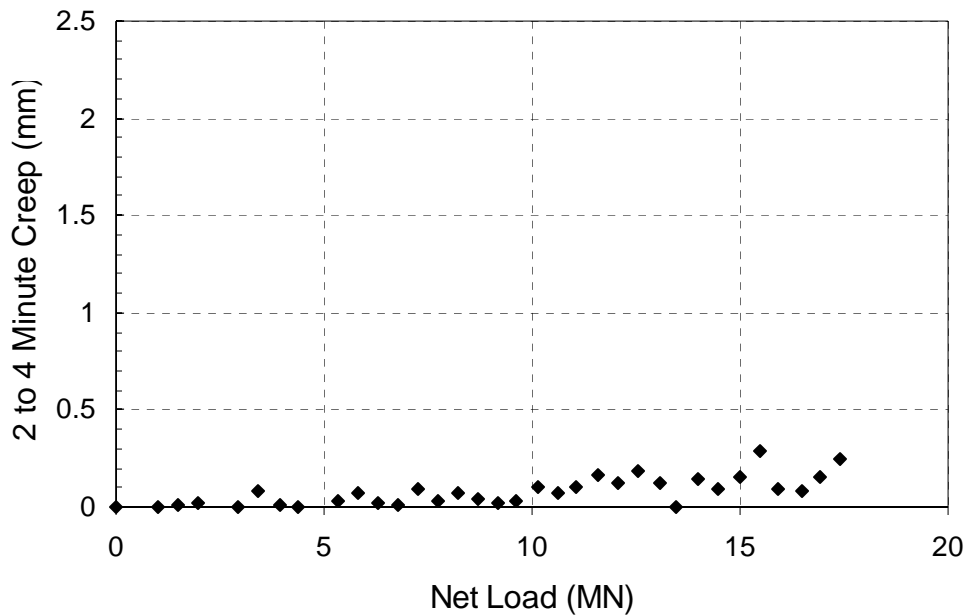
Figure 4.27- Creep displacement for the lower portion of test shaft TS-1A.

4.8.2 Test Results for TS-2

4.8.2.a Stage 1 Results: In stage 1, the lower O-cell[TM] was pressurized to

determine the combined end bearing and side shear below the O-Cell[TM] using the side

shear above the cell as the reaction. The upward and downward load-displacement

curves determined from the first stage of the load test on shaft TS-2 are plotted in Figure

4.28. In this stage, the lower O-Cell[TM] was incrementally pressurized to a maximum net

load of 10.46 MN (1175 tons), which occurred at load level 1L-13. At this loading, 60.7

mm (2.39 in) of downward movement below the lower O-Cell[TM] had occurred and the

socket segment below the O-Cell[TM] had reached ultimate capacity. The lower O-Cell[TM]

was then unloaded in 5 increments and Stage 1 was concluded.

The equivalent top-down load-displacement curves for Stage 1 loading are shown

in Figure 4.29. Again, one curve shown was computed assuming a rigid shaft; the other

curve contains an adjustment for additional elastic compression that would occur in a top-

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down load test. The curve is extended out to a displacement of 50 mm (2 in) by

extrapolating the side shear response above the O-Cell^{TM} as described in Chapter 3.
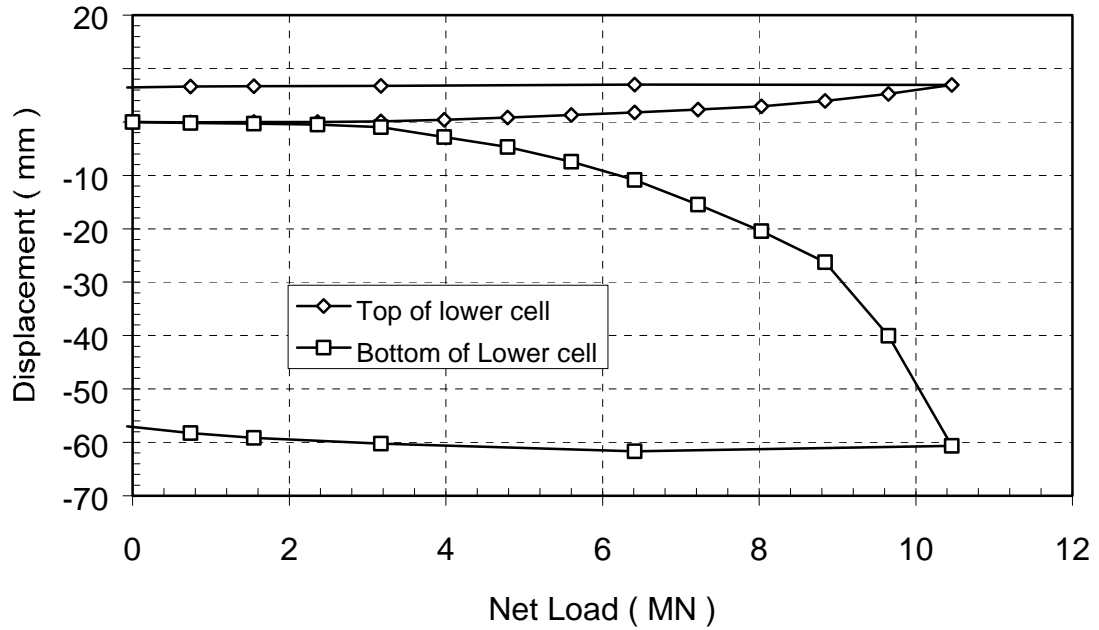
Figure 4.28- Measured load-displacement curves for lower O-cell^{TM} in test shaft TS-2, Stage 1.
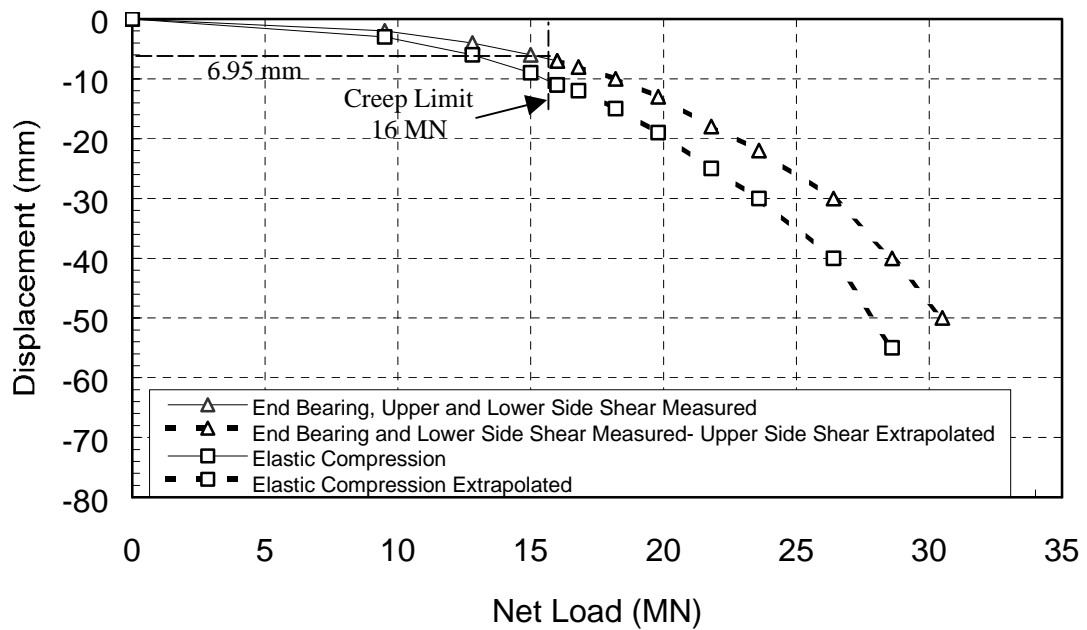
Figure 4.29- Equivalent top-down load-displacement curves for test shaft TS-2, Stage 1.

The distribution of axial force with elevation determined from the strain gage data at various loads is shown in Figure 4.30. The strain gage data used to calculate these distributions appear in Appendix A. On the day of the test, the concrete compressive strength was reported to be 33.7 MPa (4,890 psi) below elevation 178.46 m and 28.1Mpa (4070 psi) above. Elevation 178.46 m is the elevation of the cold joint. This, combined with the area of reinforcing steel and shaft diameter, was used to determine an average shaft modulus of 28.0 GPa (4060 ksi) above the upper-cell, 28.8 GPa (4170 ksi) between the upper and lower O-Cell^{TM}, and 26.9 GPa (3900 ksi) below the lower O-Cell^{TM}. The average shaft stiffness (AE) of the upper segment of shaft is 30,724 MN (6,895,000 kips), the middle segment is 30,805 MN (6,930,000 kips), and the lower segment is 25,890 MN (5,823,000 kips).

The mobilized unit side shear for each load increment is plotted versus O-cell^{TM} movement in Figure 4.31. The unit side shear curves indicate that the maximum side shear has been reached in only the shaft segment between the lower O-cell^{TM} and level 2 strain gages. The maximum side shear for this shaft segment was about 885 kPa (9.2 tsf). Very little load was transferred in side shear above strain gage level SG-2, although some load was transferred between strain gage levels SG-2 and SG-3 near the end of the test as the side shear between the O-cell^{TM} and the level 2 strain gages dropped off.
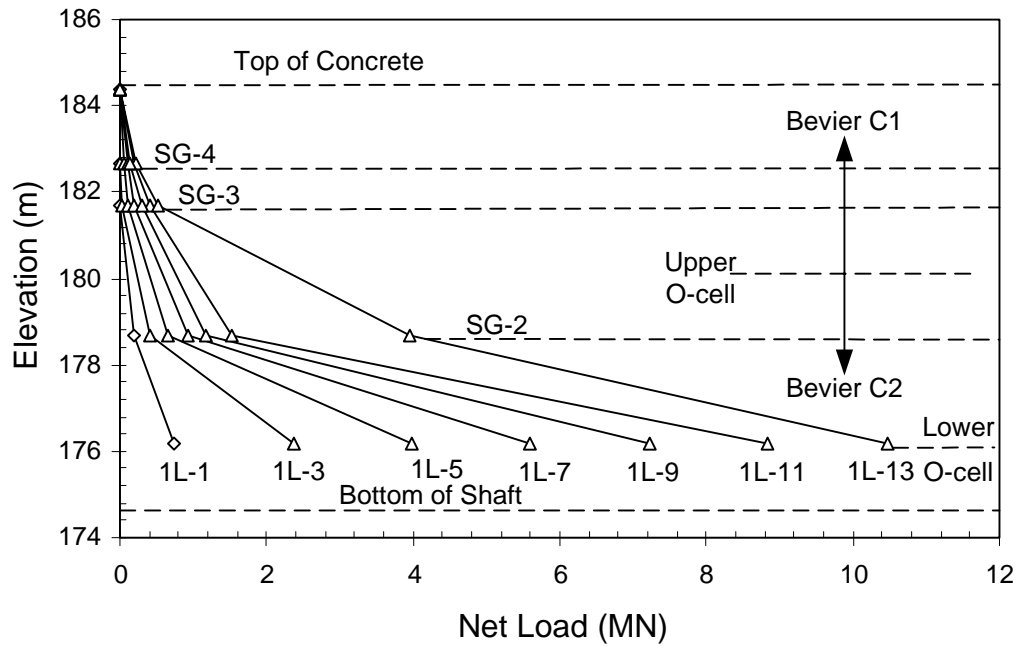
Figure 4.30-Distribution of axial force for test shaft TS-2, Stage 1.
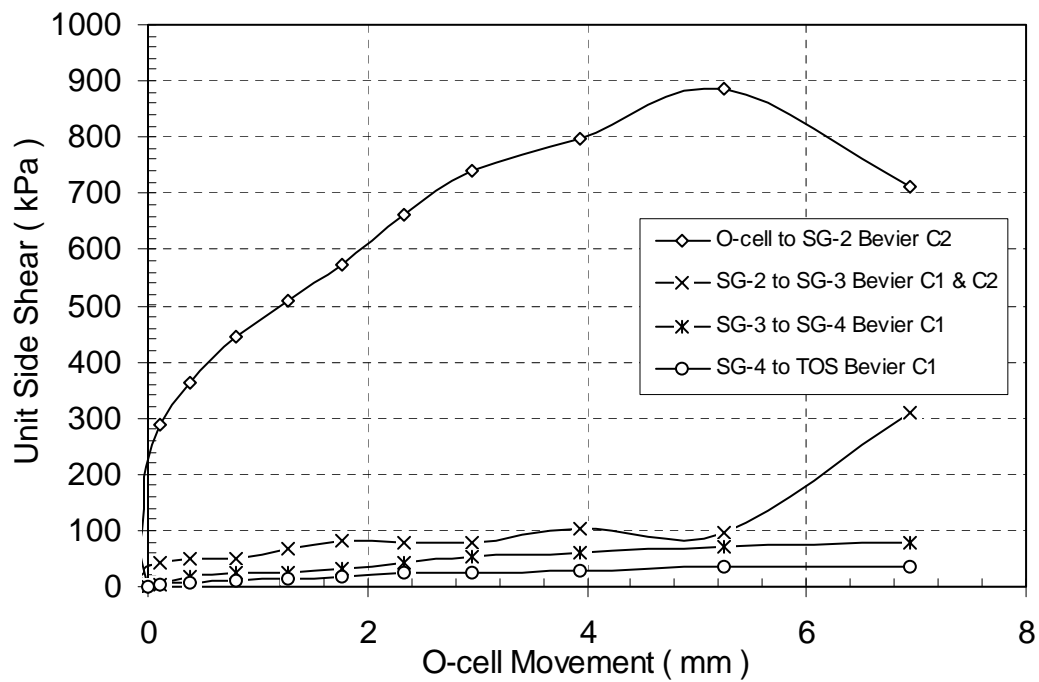
Figure 4.31- Unit side shear versus lower O-cell[TM] movement for test shaft TS-2, Stage 1.

The maximum mobilized unit side shear values determined for various segments of the shaft for Stage 1 are shown in Table 4.4 and plotted in Figure 4.32. Only the side shear value for the segment of socket between the O-cell and strain gage level SG-2 is an ultimate value. Strain gages at level SG-1 were located too close to the O-cell[TM] and gave negative readings, which indicated tension in the shaft. These readings were ignored. The unit side shear below the lower O-cell[TM] could not be determined. Assuming a unit side shear value of 918 kPa (9.6 tsf) for the segment of shaft below the lower O-cell[TM] based on values for the Verdigris Formation for shaft TS-1A, a unit end bearing value of 5826 kPa (61 tsf) may be calculated.

Table 4.4- Unit side shear values calculated from strain gage data for test shaft TS-2, Stage 1.

Load Transfer Zone	Strata	Elevation (m)	Unit Side Shear	
		From – To	kPa	tsf
SG-4 to Top of Shaft	Bevier C_1	182.67 – 184.40	34	0.36
SG-3 to SG-4	Bevier C_1	181.67 – 182.67	78	0.81
SG-2 to SG-3	Bevier C1 & C_2	178.67 – 181.67	311	3.2
Lower O-Cell to SG-2	Bevier C_2	176.17 – 178.67	885	9.2
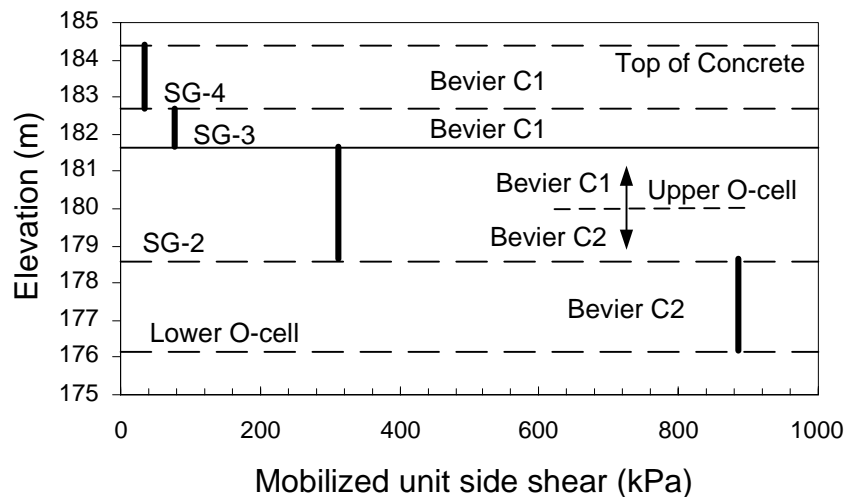
Figure 4.32- Mobilized unit side shear values calculated from strain gage data for test shaft TS-2, Stage 1.

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The displacement that occurred over the interval from 2 to 4 minutes after application of a load under a constant load is plotted versus the load in Figures 4.33 and 4.34 for the upper and lower portions of the shaft, respectively. No apparent creep limit was reached for the upper side shear for a maximum displacement of 6.95 mm (0.27 in). For the combined lower side shear and end bearing, a creep limit of 3.3 MN (370 tons) was reached at a displacement of 1.3 mm (0.05 in). The 6.95 mm displacement shown on the rigid top-down load-displacement curve in Figure 4.29 to indicates a combined creep limit of about 16 MN (1800 tons).
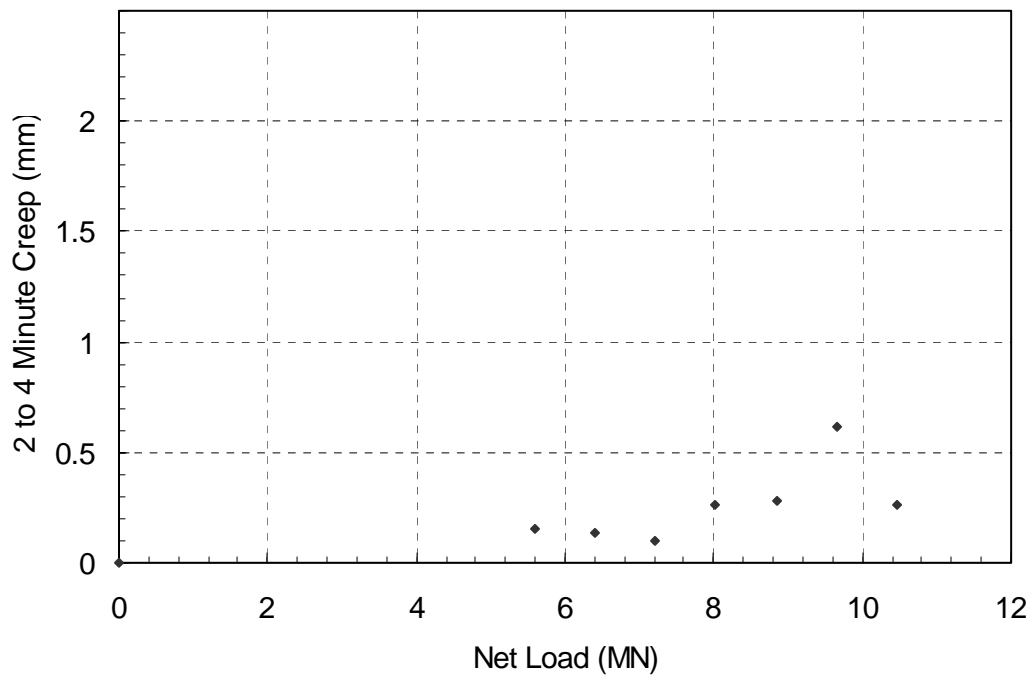
Figure 4.33- Creep displacement for the upper portion of test shaft TS-2, Stage 1.
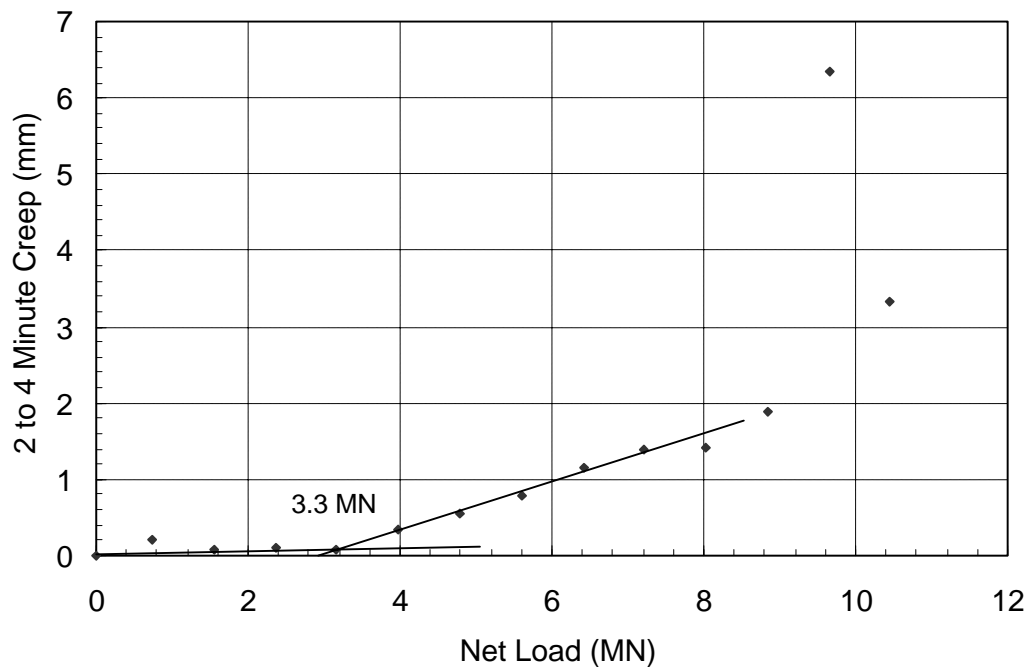
Figure 4.34- Creep displacements for the lower portion of test shaft TS-2, Stage 1.

4.8.2.b Results for Stage 2 and 3: In Stage 2, the lower O-cell[TM] was vented to serve as a compressible inclusion (zero reaction) while the upper load cell was pressurized to measure the side shear between the upper and lower cells using the side shear above the upper cell as the reaction. In Stage 3, the lower O-cell[TM] was first sealed while maintaining pressure in the upper load cell. The pressure in the upper load cell was then incrementally increased to measure the side shear above the upper load cell using the combined side shear and end bearing below the upper load cell as the reaction.

The combined upper and lower load-displacement curves for Stages 2 and 3 are shown in Figure 4.35. In Stage 2, the upper O-Cell[TM] was incrementally pressurized to a net load of 10.65 MN (1197 tons). At this load, the downward movement below the upper cell was 12.4 mm (0.49 in) and the upward movement above the cell was 3.5 mm

(0.14 in). The lower cell was closed and stage 2 was concluded.   In Stage 3,

predominantly downward movement below the upper cell continued despite closing the

lower O-Cell[TM]. The socket segment above the upper cell did not reach ultimate capacity

before the upper cell reached its maximum extension and depressurized. At the final load

of 13.3 MN (1495 tons), upward movement was 7.7 mm (0.31 in) while downward

movement was 140.4 mm (5.5 in). The equivalent top-down load-displacement curve is

shown in Figure 4.36.   The curve is extended out to a settlement of 13.1 mm (0.52 in) by

extrapolating the side shear data above the upper cell.
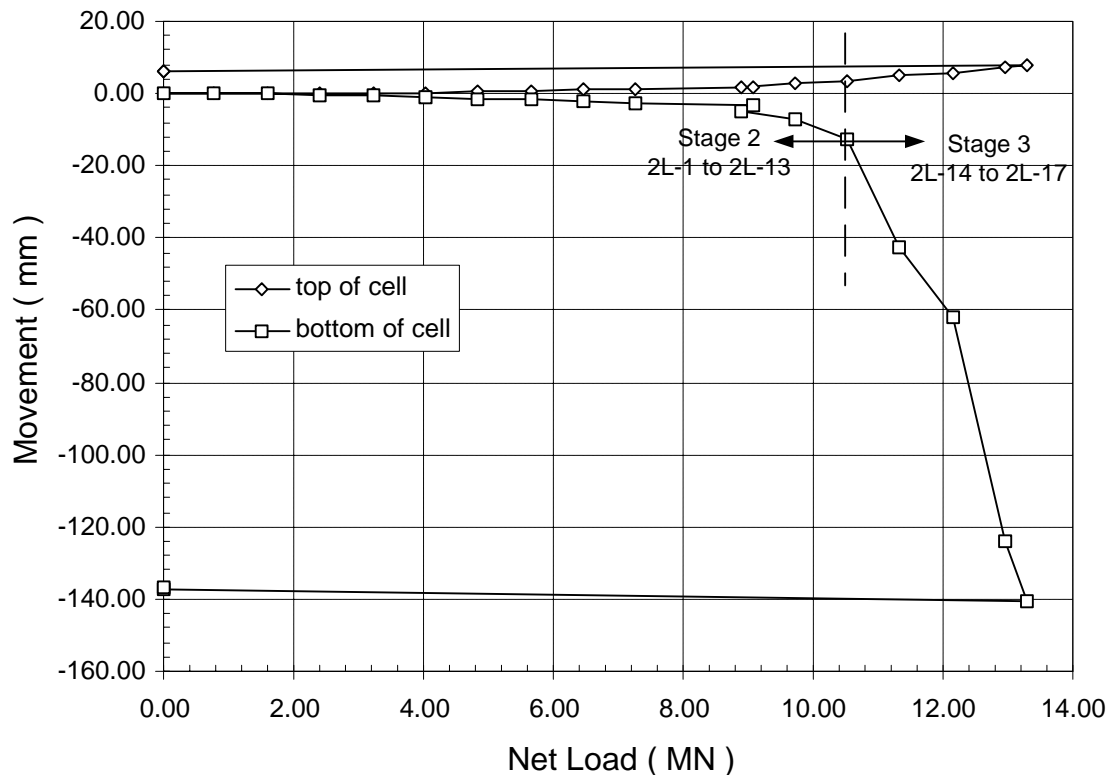
Figure 4.35- Measured load-displacement curves for downward and upward loading of
upper O-cell[TM] in test shaft TS-2, Stages 2 & 3.
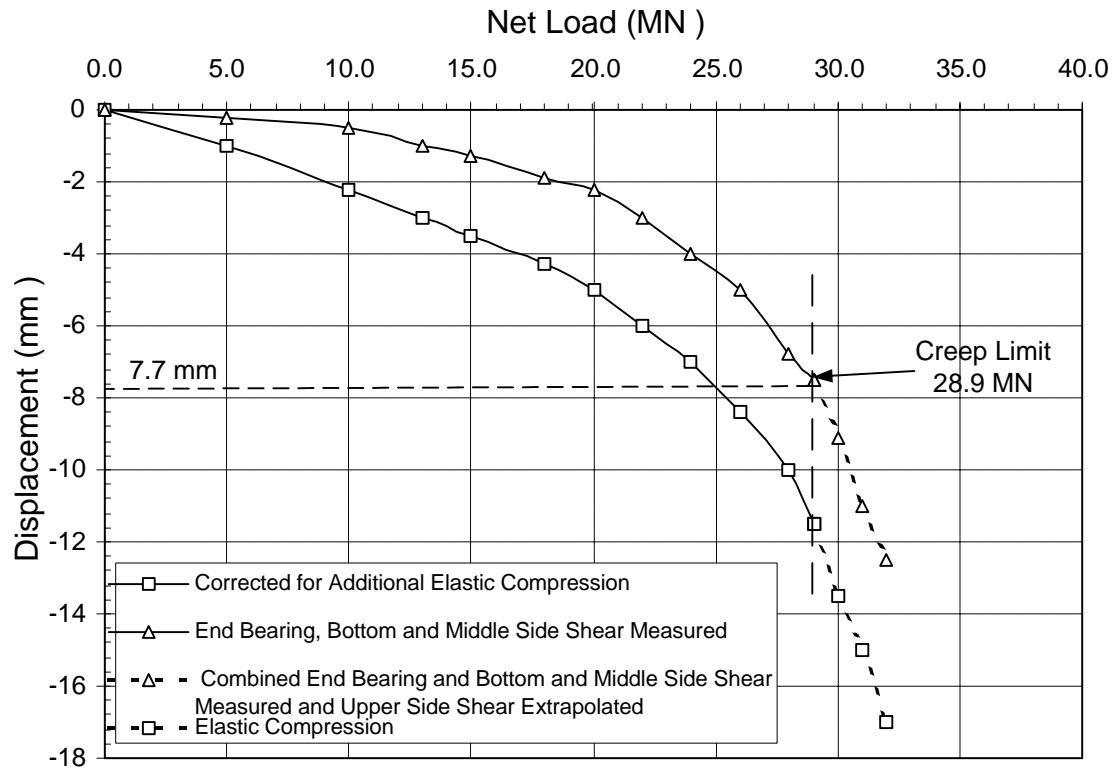
Figure 4.36- Equivalent top-down load-displacement curves for test shaft TS-2, Stages 2 and 3.

The distribution of axial force with elevation at various load increments is shown in Figure 4.37. The strain gage data is presented in Appendix A.   The values of average shaft stiffness (AE) used to determine the distribution of axial force were 30,724 MN (6,895,000 kips) for the upper segment of shaft, 30,805 MN (6,930,000 kips) for the middle segment of shaft, and 25,890 MN (5,823,000 kips) for the lower segment of shaft.

The mobilized unit side shear versus load determined for loading in Stages 2 and 3 are plotted as a function of O-cell[TM] displacement in Figure 4.38. Strain gages at levels SG-1 and SG-2 were affected by layers of questionable concrete and their proximity to the cold joint (Loadtest 1999). As a result, the strain gage data at these levels was ignored and the mobilized unit side shear in the middle section of the shaft was

determined from the relative pressures in the upper and lower O-cells[TM]. Based on the

data shown in Figure 4.38, only the segment of shaft between the upper and lower O-

cells[TM] clearly reached an ultimate unit side shear, although all the curves appear to near

ultimate values at the end of loading. Values of mobilized unit side shear determined for

the final load increment for the shaft segments are shown in Table 4.5 and plotted in

Figure 4.39. The socket segment between the O-Cells[TM] reached an ultimate unit side

shear of 726 kPa (7.6 tsf). The average measured unit side shear in the 4.23 m (13.9 ft)

shaft section above the upper O-Cell[TM] was 846 kPa (8.8 tsf). An average ultimate unit

side shear value of 976.8 kPa (10.2 tsf) was determined for shaft section above the upper

O-Cell[TM] by extrapolating and curve fitting the upper load displacement curve.
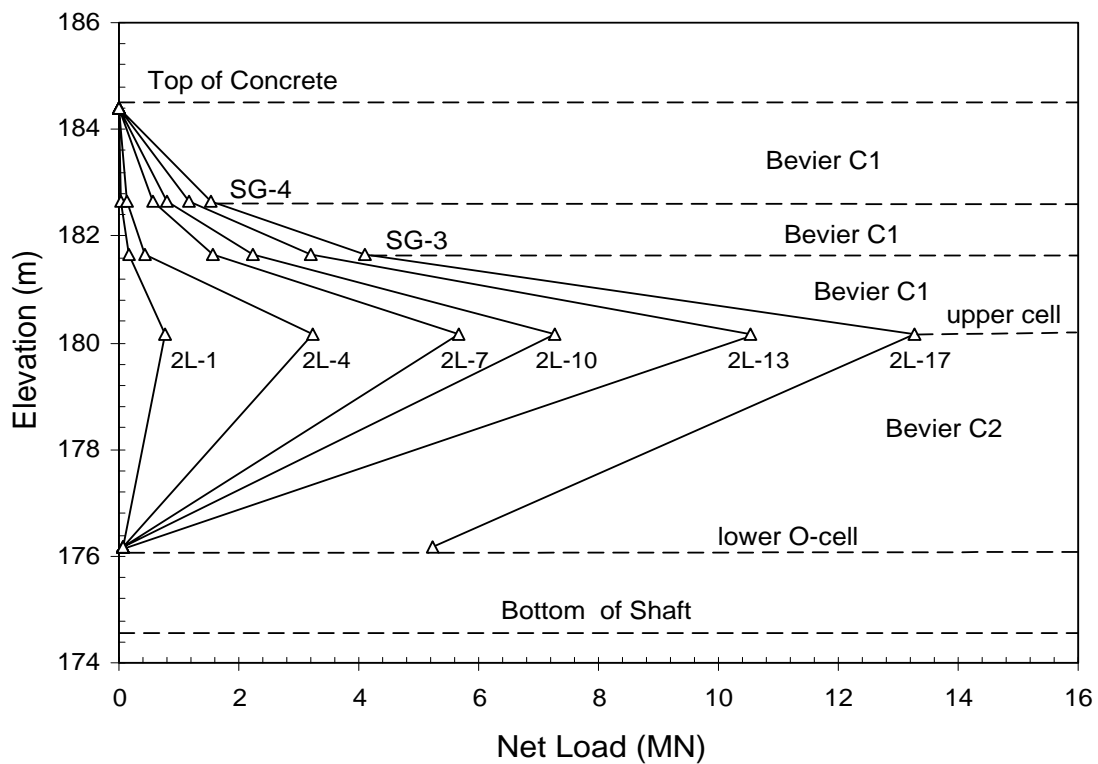
Figure 4.37- Distribution of axial force for test shaft TS-2, Stages 2 & 3.

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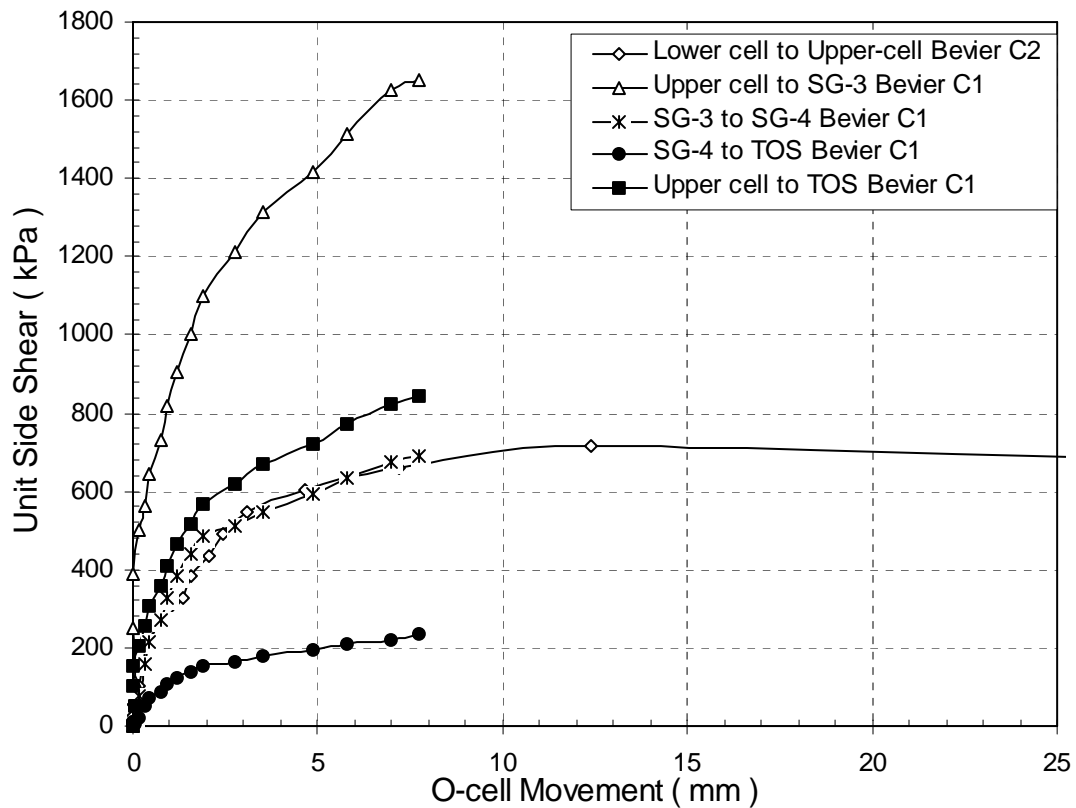
Figure 4.38-Unit side shear versus upper O-cell^{TM} movement for test shaft TS-2, Stages 2 & 3.

Table 4.5-Unit side shear values calculated from strain gage data for test shaft TS-2, Stages 2 & 3.

Load Transfer Zone	Strata	Elevation (m)	Unit Side Shear	
		From – To	kPa	tsf
upper cell to Top of Shaft	Bevier C_1	180.17 to 184.40	846	8.8
SG-4 to Top of Shaft	Bevier C_1	182.67 – 184.40	236	2.45
SG-3 to SG-4	Bevier C_1	181.67 – 182.67	694	7.3
upper cell to SG-3	Bevier C_1	180.17 – 181.67	1653	17.3
lower to upper cell	Bevier C_2	176.17 - 180.17	726	7.6

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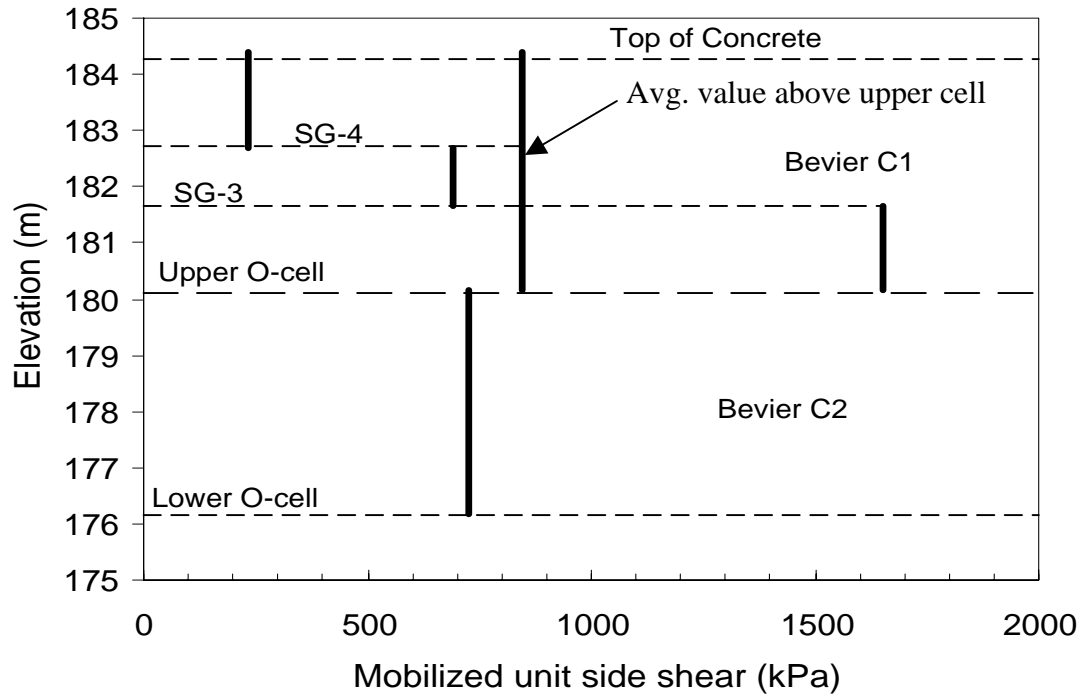
Figure 4.39- Mobilized unit side shear values calculated from strain gage data for test shaft TS-2, Stages 2 & 3.

Figures 4.40 and 4.41 show the creep displacements during stages 2 and 3 for the upper and middle portions of the shaft as a function of net loads. A creep limit was not reached for the upper side shear at a maximum displacement of 7.7 mm (0.31 in) during stages 2 and 3. The middle side shear creep data indicated that a creep limit of 8.0 MN (900 tons) was reached at a displacement of 2.9 mm (0.12 in) during Stage 2. The combined end bearing and lower side shear data from Stage 1 indicated that a creep limit of 3.3 MN (370 tons) was reached at a displacement of 1.3 mm (0.05 in) for the lower segment of the shaft. Since a top-loaded shaft will not begin to creep significantly until all the components begin creep movement, significant creep will not begin until top loading exceeds 28.9 MN (3245 tons) at a displacement of 7.7 mm (0.31 in) by some unknown amount (Figure 4.36).

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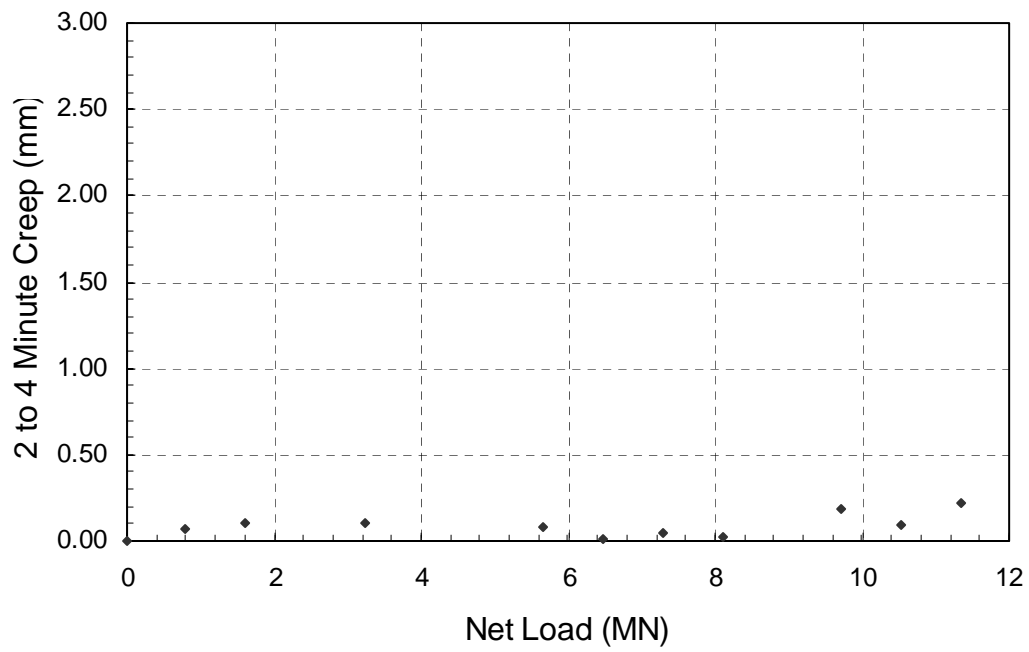
Figure 4.40- Creep displacement for upper segment of shaft for test shaft TS-2, Stages 2 & 3.
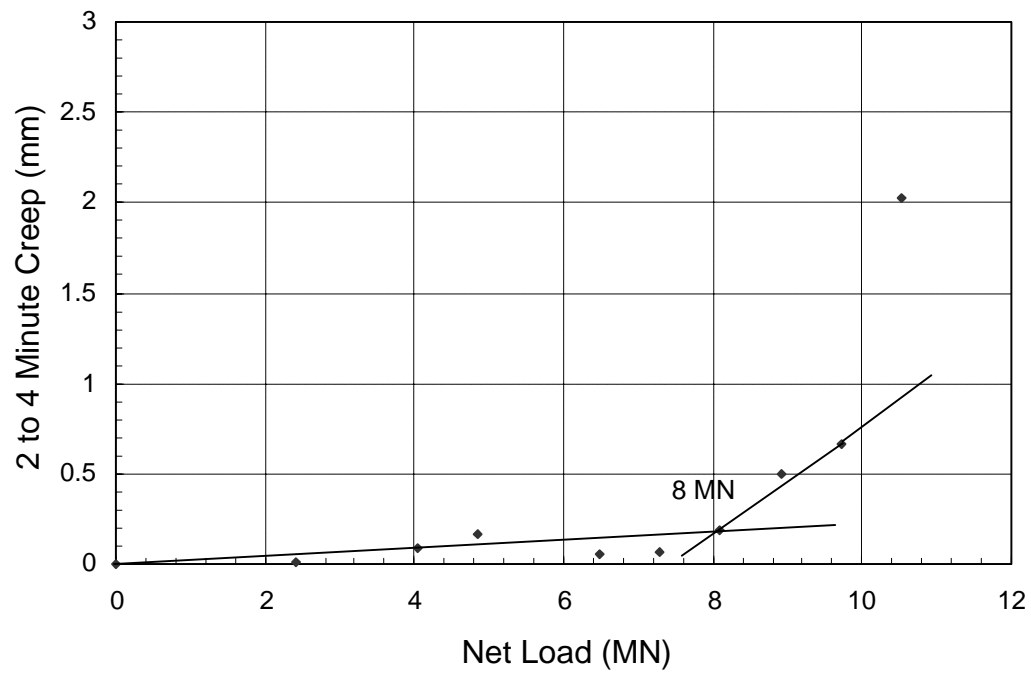
Figure 4.41- Creep displacement for middle segment of shaft for test shaft TS-2, Stage 2.

4.9 Practical Applications

The Osterberg cell load tests where successful in allowing MoDOT to develop a more economical design for the drilled shafts for the proposed bridge alignment. Table 4.6 shows a cost break down for the two Osterberg cell load tests and Table 4.7 shows the anticipated cost savings. The anticipated costs savings were based on the drilling costs of the rock socket and did not include shaft concrete or reinforcing steel. There would also be a reduction in the amount of cross-hole sonic logging and verification coring of selected shafts. Finally this cost did not include additional geotechnical investigation that would be required if the rock sockets could not be shortened. The additional geotechnical investigation would cost about $150,000.

Table 4.6- Costs of Osterberg cell load tests for test shafts TS-1A and TS-2.

Mobilization	$45,000	
Drilling Overburden	$89,000	46 m (12220 mm Dia.)
Steel Casing	$16,000	48 m (1220 mm ID)
Drilling Rock Socket	$63,000	32.5 m (1067 mm Dia.)
Drilled Shaft Concrete	$ 2,000	14.3 Cu. Meter
Load Test (one cell)	$75,000	
Load Test (two cells)	$142,000	
Design Cost	$60,000	

Table 4.7- Anticipated cost savings for drilled shafts.

Pier	Length of Socket		Length	No. of	Amount
	Before Test	After Test	Saved	Shafts	Saved
	(m)	(m)	(m)		
19	25.6	7.3	18.3	6	$360,250
20	31.0	6.9	24.1	8	$632,580
21	23.2	7.5	15.7	15	$772,680
22	21.2	14.8	6.4	8	$167,990
23	16.8	8.5	8.3	6	$163,390
24	21.1	10.2	10.9	6	$214,580
				Total	$2,311,470
				Test Costs	-$492,000
				Cost Savings	$1,819,470

4.10 Summary and Conclusions

The Missouri DOT proposed to build a new bridge across the Missouri River in central Missouri. The proposed foundation design for the piers in the vicinity of the river consisted of drilled shafts socketed into bedrock. The bedrock at this location consisted of alternating layers of clay shale, siltstone, coal, and underclay with scattered layers of limestone and sandstone. Since the shales could not support large axial loads in end bearing it was decided to design the rock sockets based side shear only. Current design methods used by MoDOT would require exceedingly long rock sockets that would be very expensive. In order to reduce costs and to better quantify the available unit side shear capacity of bedrock, two Osterberg cell load tests were performed in May and June of 1999 between periods of high water.

A general geologic description of the Lexington test site was presented in this chapter followed by a summary of the engineering characteristics of the most pertinent strata. The construction and testing procedures for the two test shafts were described, followed by presentation of the results from each load test. The values of unit side shear determined from the Osterberg cell load tests exceeded the anticipate values and allowed the shafts to be designed more economically.

CHAPTER FIVE
GRANDVIEW TRIANGLE TEST SITE

The Missouri Department of Transportation (MoDOT) is currently in the process of "untangling the triangle" in metropolitan Kansas City, Missouri. The intersection, known as the Grandview Triangle, is located in southern Jackson County and handles about 250,000 vehicles per day on three major interstate routes and 7 local routes. The 250 million dollar make over of the triangle is to be phased in over 7 to 8 years with the total replacement of all bridges and pavement. The completed project is projected to accommodate more than 400,000 vehicles per day and increase the size of the triangle from the present 284 acres to more than 376 acres. All work is being done while maintaining current traffic volumes.

The construction of new bridges and walls alongside the existing bridges has presented many challenges to bridge and geotechnical engineers. A twelve span structure (A6252) and a thirteen span structure (A6254) will cross US 71 and will require the construction of footings in the median of US 71. The use of conventional spread footings or pile caps would require a detour to allow room for the footing excavation. Drilled shafts socketed into bedrock would not require a detour and were investigated as the preferred foundation type. Previously constructed drilled shafts in the triangle have been assumed to carry axial load in end bearing only. Since the closest limestone layer thick enough to support the axial loads in end bearing only is about 80 feet (24.4 m) below the top of the shaft, an Osterberg cell load test was performed to investigate the unit side shear characteristics of the bedrock to determine if adequate axial capacity could be achieved in shallower strata by considering side shear only.

In this chapter, the general conditions at the site are described followed by descriptions of the engineering characteristics of the soil/rock of most importance to this site. The construction and loading of the test shaft is then described, followed by presentation of the results from the load test. The reconstruction of the Grandview Triangle was designed using English units. All results are therefore reported using English units or dual units.

5.1 Site Description

The Grandview Triangle site is located in metropolitan Kansas City, Missouri. The area is known to local residents as the Grandview Triangle because three major interstate routes (US71, I-470, and I-435) and 7 local routes converge in this area. A map of the southern Kansas City metropolitan area is shown in Figure 5.1. A plan view of the overall project is shown in Figure 5.2 and the location of the test shaft is shown in Figure 5.3.

The project is situated on moderate to steeply sloping rolling hills that border Hickman Mills Creek. The project soils originated from residuum of the weathering of limestone and shales. About 70 per cent of the ground surface is covered by Snead soils of the Snead-Urban land complex. The remaining 30 per cent is covered by the Urban land portion of the complex, which consists of residential and commercial development including roadways and bridges. The Liquid Limits ranged from 40 to 64 for the Snead soil. The Plasticity Index (PI) varied from 18 to 38 and the Snead is predominately a CH soil by ASTM classification.
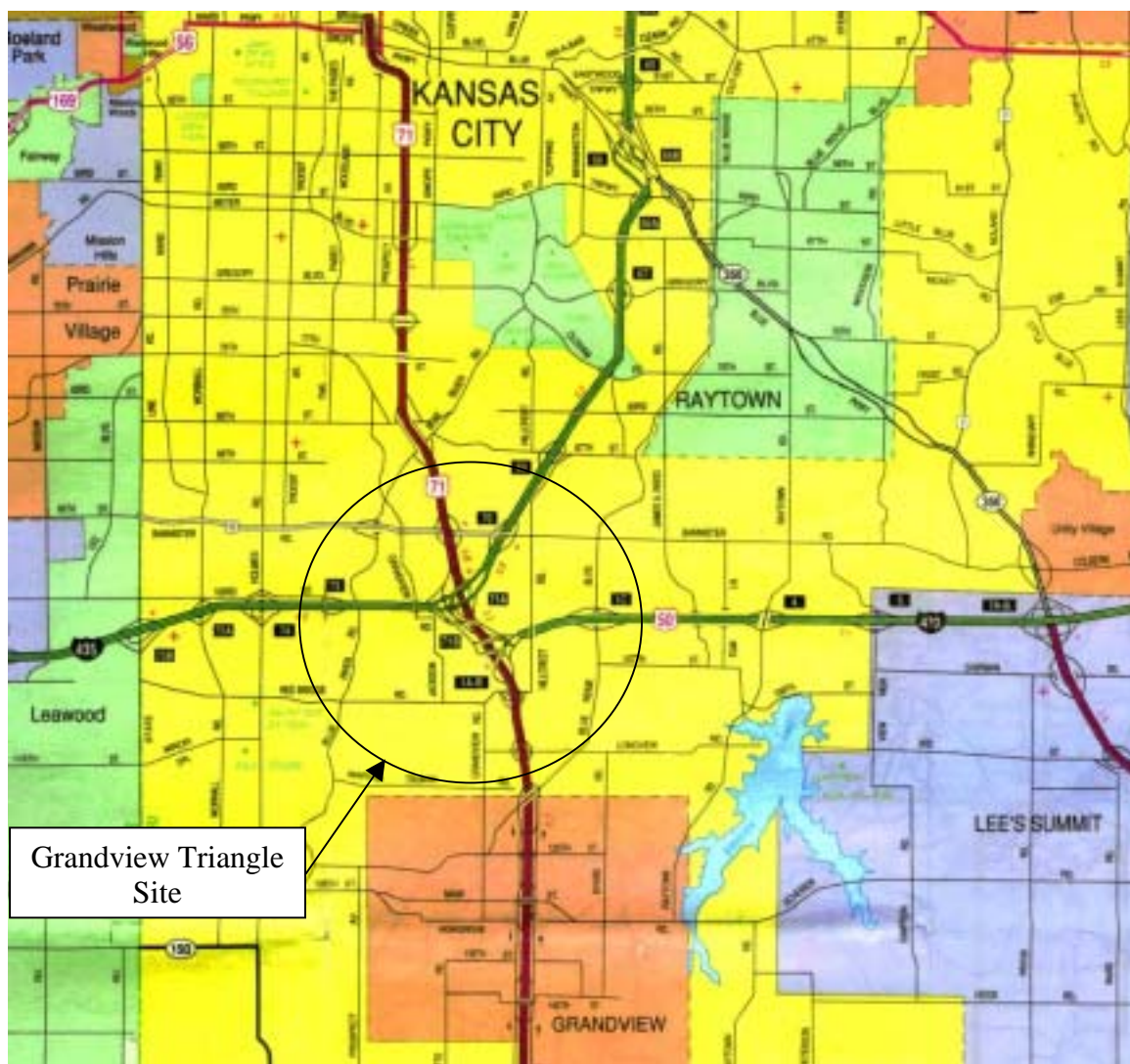
Figure 5.1- Roadmap of Grandview Triangle area in Metropolitan Kansas City.
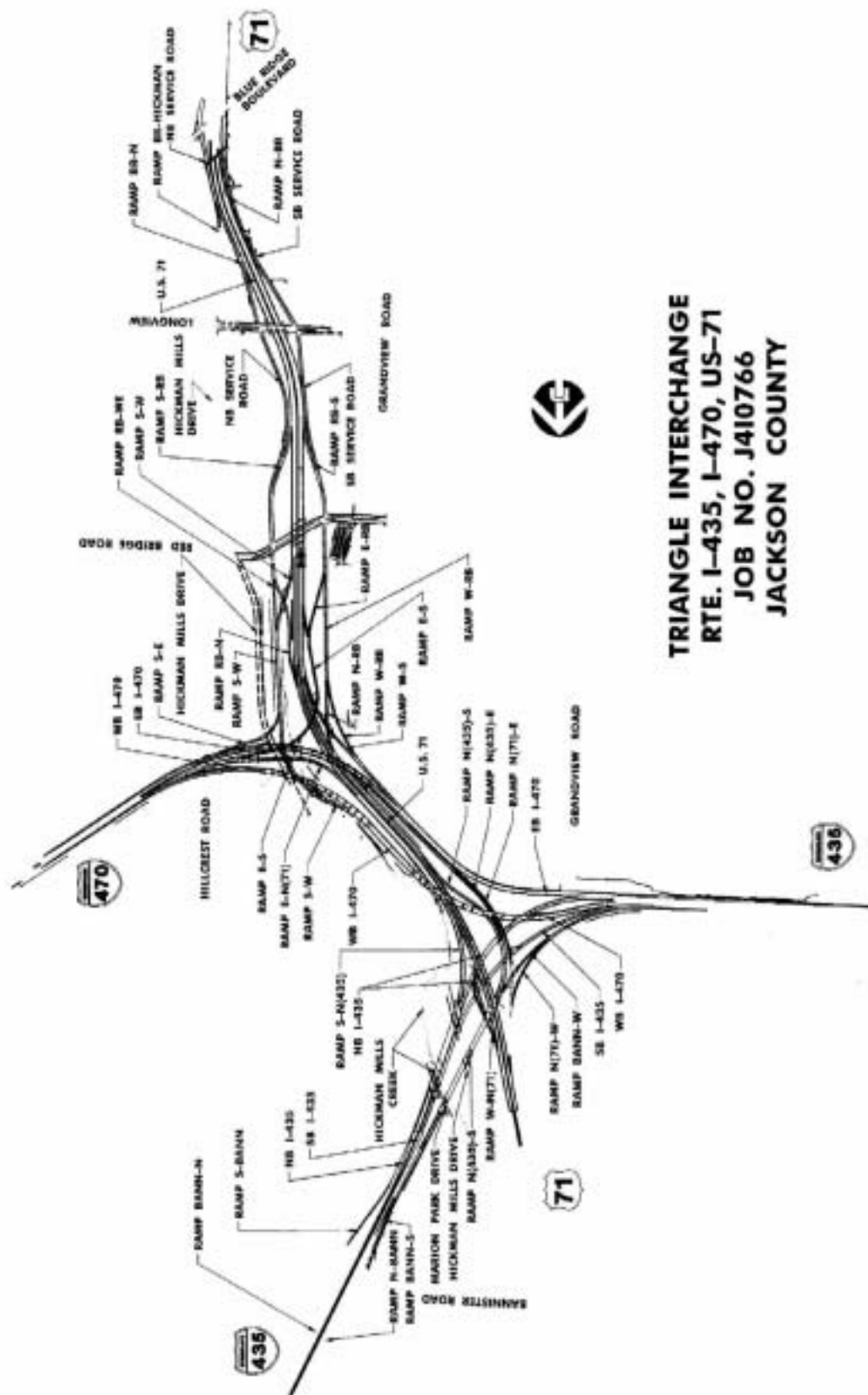
Figure 5.2 Grandview Triangle (from HNTB 2002).

Figure 5.3 Grandview Triangle bridges and Osterberg cell load test site (from HNTB 2000).
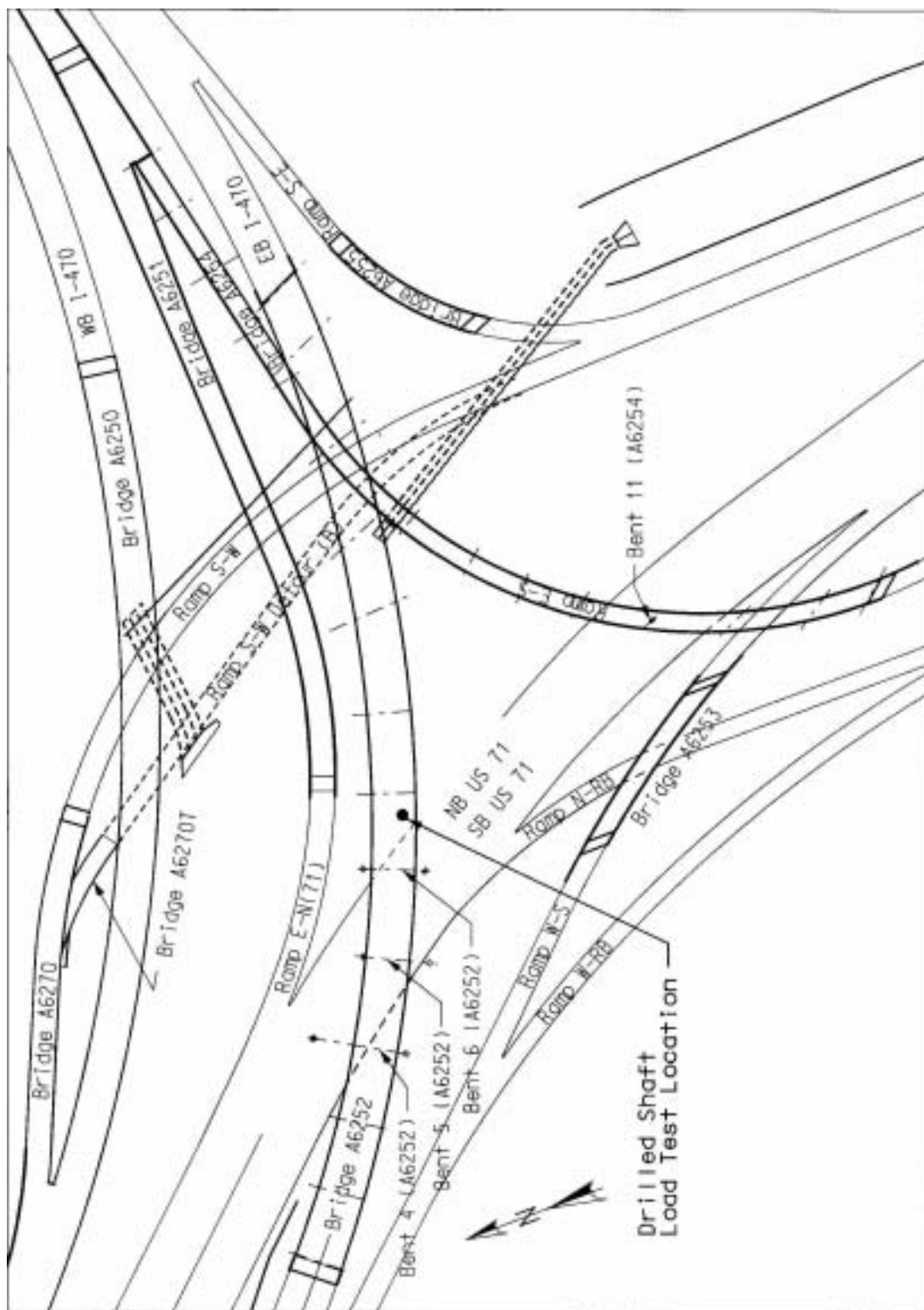
Geologic units of the Kansas City Group are exposed within the limits of the project. The Kansas City Group is a Pennsylvanian Age deposit consisting predominately of alternating sequences of horizontally bedded limestones and shales as shown in Figures 5.4 and 5.5 (in descending order). Shale units may be very hard to calcareous in nature or clay-like in the form of claystones and siltstones. Geologic units exposed in various road cuts and encountered during the subsurface investigation in descending order varied from the Argentine Limestone Member of the Wyandotte Formation to the Bethany Falls Limestone Member of the Swope Formation. The Argentine Limestone was encountered at about elevation 973.7 ft. and the Bethany Falls Limestone was encountered at about elevation 826 ft.

All of the existing bridges, ramps, and pavement at the site will be replaced with 25 new bridges and more than 40 retaining walls. The bridges will consist of simple 3 span plate girder structures to large multiple span structures. Foundations for most bridges in the project area include drilled shafts and spread footings. Drilled shafts for most of the large bridge structures were socketed into either the Winterset Limestone or the Bethany Falls Limestone. These shafts were designed to carry axial load in end bearing only with an allowable end bearing pressure of 30 to 40 tsf (2.87 to 3.83 MPa). However, for bridges A6252 and A6254 shown in Figure 5.3, these strata were excessively deep and, since several piers were located in the median of US 71, spread footings would have required excessive traffic closures. An O-cell test was therefore planned to evaluate if the design load could be supported in side shear from shallower strata. A more detailed geologic description of the Osterberg cell load test site is presented in the following section.

GEOLOGIC COLUMNAR SECTION IN THE KANSAS CITY AREA - URS CORPORATION

GROUP	FORMATION	MEMBER	COLUMN	THICKNESS AVG.	RANGE	No.
PECE	WESTON SHALE				5' ±	1
	STANTON	SOUTH BEND LIMESTONE		4'	3.5' - 5'	2
		ROCK LAKE SHALE		8' ±	3.4' - 12'	3
		STONER (OLATHE) LIMESTONE		14'	11' - 16.5'	4
LANSING GROUP	LIMESTONE FORMATION	EUDORA SHALE		6'	4' - 7.3'	5
		CAPTAIN CREEK LIMESTONE		5'	3' - 6.6'	6
	VILAS SHALE			17'	9' - 24.4'	7
	PLATTSBURG LIMESTONE	SPRING HILL LIMESTONE		13'	10.4' - 16'	8
		HICKORY CREEK SHALE		1'	0.3' - 1.9'	9
		MERRIAM LIMESTONE		2'	1.1' - 4.0'	10
KANSAS CITY GROUP — ZARAH SUB-GROUP	BONNER SPRINGS SHALE			27'	14' - 37.2'	11
	WYANDOTTE LIMESTONE FORMATION	FARLEY LIMESTONE		21'	10' - 29.8'	12
		ISLAND CREEK SHALE		9'	1.2' - 30'	13
		ARGENTINE (CRUSHER LEDGE) LIMESTONE		25'	20' - 40'	14
		QUINDARO SHALE		1'	0.2' - 7'	15
		FRISBIE LIMESTONE		2'	4' - 5'	16

DESCRIPTION

1) Weston Shale - Blue-gray, weathers gray-green, clay shale.

2) South Bend Limestone- Dark gray fine-grained limestone, sandy in lower part, weathers tan, some thin shale partings.

3) Rock Lake Shale- Gray, some red locally, sandy, platy to blocky, locally contains a very hard lime cemented sandstone of variable thickness and position; other sandstones may be present.

4) Stoner Limestone- Light bluish-gray to nearly white, mostly thin wavy bedded, with thin discontinuous shale partings, fine grained with crystalline calcite locally, weathering produces prominent joint cracks.

5) Eudora Shale - Dark gray and black fissile shale in lower one-third; greenish-gray to bluish-gray in upper two-thirds; commonly 1' to 2' blocky shale underlain by 1' of dark blue shale, water bearing.

6) Captain Creek Limestone- Dark gray to bluish gray, weathers tan, granular or dense, brittle, massive or even bedded, vertical joints fairly prominent, upper few inches may be brecciated, silicified, mottled pink and gray bed. The average thickness of 5' is fairly consistent.

7) Vilas Shale - Variable. Gray to buff, with some green in lower part, clayey to sandy shale. Shaly limestone may be present in the lower part, sandstones may occur in variable amounts, position and degree of cementation.

8) Spring Hill Limestone- Light gray oolitic or granular medium bedded limestones in upper part and bluish-gray to light gray, fine grained, brittle, thin-bedded limestone in lower part. Weathers tan and yellow. Usually has several limy shale partings and locally has chert in lower part.

9) Hickory Creek Shale- Green, gray, or reddish-orange clayey or limy shale, which may contain a black fissile carbonaceous zone.

10) Merriam Limestone - Light gray to blue-gray unit bedded to slabby limestone, weathers tan.

Note #1 - Presence of Materials above the Bonner Springs Shale is rare east of the Kansas-Missouri State Line.

11) Bonner Springs Shale- Highly variable. Gray to buff clay shale, sandy shale and sandstone. The upper part is olive-green, blocky shale, often with a maroon band near the top. Several thin soft impure nodular limestones are usually present near the upper contact. Lensing cross-bedded sandstones and sandy shale may be present, usually in the middle portion. The lower part is olive platy clay shale, and may contain a thin limestone near the base.

Note #2 - Bonner Springs, Farley and Island Creek Members Total 50 to 60 Feet.

12) Farley Limestone- Highly variable. Usually two beds of limestone separated by shale. The upper limestone is light gray to gray, dense, often unit bedded where thin, and generally massive where thick, weathers tan or orange-tan. The shale is olive, blue-gray or dark gray, weathers tan. May contain thin impure limestones and usually has a sandstone and sandy shale zone near the middle part. The lower limestone is light gray to blue-gray, thin wavy bedded, weathers light gray to tan. It is similar to the Argentine limestone below and has much secondary porosity due to solutioning along bedding planes and joints.

13) Island Creek Shale- Highly variable. Dark gray, olive, yellow and/or bluish clay shale. May be slightly blocky, may be sandy, sometimes contains thin sandstone zones. Locally in Wyandotte County several feet of channel sandstone occupies this position. The thickness in Clay and Platte Counties is greater than in Jackson County.

14) Argentine Limestone- Variable in thickness. Light bluish-gray, wavy thin bedded limestone, weathers nearly white. Sometimes has massive bed at top, usually has non-persistent shale parting. Has secondary porosity similar to the lower ledge of the Farley. Commonly has mud seams, cavities, small vuggs and secondary calcite. Rarely it contains chert nodules.

15) Quindaro Shale - Black fissile and yellowish-gray limy shale, or fairly soft gray shaly limestone which in outcrops undercuts the overlying Argentine, but is difficult to detect in cores.

16) Frisbie Limestone - Bluish-gray to dark blue massive limestone, weathers light gray to buff.

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Figure 5.4 Kansas City Stratigraphy (from URS 2001).

GEOLOGIC COLUMNAR SECTION IN THE KANSAS CITY AREA

GROUP	FORMATION	MEMBER	COLUMN	THICKNESS (FT) AVG.	RANGE	No.
ZARAH	LANE SHALE			18'	7' - 70' +/-	17
	IOLA LIMESTONE	RAYTOWN (CALICO LEDGE) LIMESTONE		6'	5.2' - 8'	18
		MUNCIE CREEK SH		2'	0.1' - 5'	19
		PAOLA LIMESTONE		2'	0.9' - 2.8'	20
	CHANUTE SHALE			19'	14' - 24.3'	21
LINN SUB - GROUP	DRUM (BUILDING LEDGE) LIMESTONE	CEMENT CITY LIMESTONE		9'	4' - 13.5'	22
	CHERRY-VALE SHALE FORMATION	QUIVIRA SHALE		6'	3' - 26'	23
		(REEF LEDGE) WESTERVILLE (OOLITIC LEDGE) LIMESTONE (BULL LEDGE)		12'	3' - 23.2'	24
		WEA SHALE		21'	14' - 30'	25
		BLOCK LIMESTONE		6'	0.6' - 7.9'	26
		FONTANA SHALE		5'	1' - 13'	27
KANSAS CITY GROUP / BRONSON SUB - GROUP	DENNIS LIMESTONE FORMATION	WINTERSET (CHERT LEDGE) LIMESTONE		29'	26' - 35'	28
		STARK SHALE		3.5'		29
	GALESBURG SHALE			3' ±	2.5' - 9'	30
	SWOPE LIMESTONE FORMATION	BETHANY FALLS (CEMENT ROCK) LIMESTONE		20'	15' - 26'	31
		HUSHPUCKNEY SHALE		4'	2' - 7'	32
		MIDDLE CREEK LIMESTONE		3'	1.4' - 5.3'	33
	LADORE SHALE			4'	1' - 15'	34
	HERTHA LIMESTONE	SNIABAR LIMESTONE		3'-14'		35
		MOUND CITY SH		3'-4'	3.4' - 13.8'	36
		CRITZER LS.		0'-3'		37

DESCRIPTION

17) Lane Shale - Dark bluish-gray shale and gray and yellowish-brown sandy shale. Weathers light green to tan or yellow. It contains a channel sandstone near Hickman Mills. The Lane is thin in the southern part of Kansas City, thickens in the north part of the city and in southern Jackson County. It also thins to the east of the city but thickens near Lees Summit.

18) Raytown Limestone - The color is irregular gray, blue, buff and reddish, the texture is quite variable. The limestone is wavy bedded to massive and may have shale breaks. Weathering causes stains and blotches and produces a rough surface.

19) Muncie Creek Shale - Gray or gray-green clay shale in the upper and black fissile shale in the lower part. The shales are often separated by a thin shaley limestone. Water bearing.

20) Paola Limestone - Dark bluish-gray unit bedded limestone, weathers bluish-gray.

21) Chanute Shale - Variable. Dark gray to green, platy to blocky shale and/or yellowish brown sandy shale and highly variable sandstone (usually in the upper portion). A red or maroon zone occurs locally near the middle. The lower portion is usually lighter in color and may be limy. Weathers to a fat clay.

22) Drum Limestone - Blue-gray to gray green, massive to slabby limestone. Weathers light gray to tan to nearly white with deep yellow staining along bedding plains and joints.

23) Quivira Shale - Variable. Blue-gray to olive shale, locally has about one foot of black sub-fissile or maroon shale near the middle, weathers light olive green. The combined average thickness of the Quivira Shale and underlying Westerville Limestone is 18 feet. This shale weathers to clay rapidly and is not too sound for engineering purposes.

24) Westerville Limestone - Highly variable in the upper part, which may be represented by cross bedded oolitic limestone, by shale or by an interbedded combination of the two in any proportion. Where this part is composed entirely of shale it is not distinguishable from the overlying Quivira Shale and is assigned to that member. The limestone is generally gray to blue-gray, fairly thin bedded and weathers nearly white. The separating shales are usually blue-gray and weather olive. In the Parkville area the upper limestone is cherty. The complete range of variation in this zone can take place within a very short distance. The lower part is a very consistent light gray, usually unit bedded limestone which ranges from 2 to 6 feet in thickness but averages 3 to 4 feet.

25) Wea Shale - Dark blue-gray to gray green shale, locally may be subfissile, may have a thin zone of maroon silty shale in the upper part, weathers olive. It usually has several thin impure limestone or siltstone zones in the lower few feet. This shale is usually wet and weathers rapidly to fat clay.

26) Block Limestone - Variable. Usually two blue-gray limestones separated by blue-gray shale. The lower limestone is consistent, averaging 1 to 2 feet in thickness. The upper bed may be absent locally.

27) Fontana Shale - Dark blue-gray shale, may have a black sub-fissile zone in the middle or lower part. Weathers gray-green to buff.

28) Winterset Limestone - The upper part is blue gray to dark gray slabby to massive limestone with abundant nearly black chert nodules and numerous irregular shale breaks. A 1 to 3 foot very dark gray shale is present near the middle. The lower part is light gray irregularly bedded limestone with some light gray chert and shale seams.

NOTE #3 - The Canville Limestone, lowest member of the Dennis Limestone Formation, is usually absent in the Kansas City Area.

29) Stark Shale - Gray and yellow shale in the upper and black fissile shale in the lower part. This shale is water bearing.

30) Galesburg Shale - An under-clay like relatively structureless soft light gray nodular calcareous shale.

31) Bethany Falls Limestone - Except for the uppermost 1 to 3 feet, which is commonly a fairly soft, clayey, nodular limestone similar to the overlying Galesburg shale, the upper part is gray, mottled, massive algal or nearly white oolitic limestone which may be cross bedded. A fairly thin shale bed may be present locally near the middle. The lower portion is light gray, dense, thin bedded limestone. The entire unit weathers nearly white. The 20 foot thickness is fairly consistent, but the bed attains a maximum of 26 feet southeast of Independence. Vertical joints are prominent and outcrops usually exhibit large detached rocks.

32) Hushpuckney Shale - Bluish-gray clay shale in the upper and black fissile shale in the lower part. This shale is water bearing.

33) Middle Creek Limestone - One or two dark bluish-gray, vertical joints.

34) Ladore Shale - Variable. Gray to buff clayey to sandy shale. May locally be replaced by algal limestone which is assigned to Sniabar.

35) Sniabar Limestone - 1 or 3 beds of gray limestone. Weathers brown.

36) Mound City Shale - Gray calcareous shale in the upper and dark calcareous in the lower part.

37) Critzer Limestone - Brownish-gray, nodular, impure limestone which grades into red or green shale.

DRAWING NO. 3

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Figure 5.5 Kansas City Stratigraphy (from URS 2001).

5.2 Geology of the Area

The underlying bedrock in the project area is of upper Pennsylvanian Age, Missourian Series, Kansas City Group. The Kansas City Group is divided into three subgroups (in ascending order): the Bronson, Linn, and Zarah. Rock sockets for drilled shafts to support Piers 4, 5, and 6 for bridge A6252 and Pier 11 for bridge A6254 are planned for location in shales and limestones of the Linn Subgroup as shown in Figure 5.6.

The Linn Subgroup consists of horizontally bedded shales and limestones. The Linn Subgroup consists of (from the base upwards): the Cherryvale Formation, the Drum Formation, the Chanute Formation, and the Iola Formation (Thompson 1995). The relevant strata from these formations are described in more detail below.

5.2.1 Cherryvale Formation

The Cherryvale Formation is composed of five members (from the base upwards): the Fontana Shale Member, the Block Limestone Member, the Wea Shale Member, the Westerville Limestone Member, and the Quivira Shale Member. The Fontana Shale Member and Block Limestone Member were significantly below the base of the rock socket and were not included in the analysis. The Fontana Shale Member was encountered from about elevation 852 to 862 ft and averages about 10 feet (3.3 m) in thickness. The Block Limestone Member was encountered from about elevation 862 to 864 ft and averages about 2 feet (0.6 m) in thickness.

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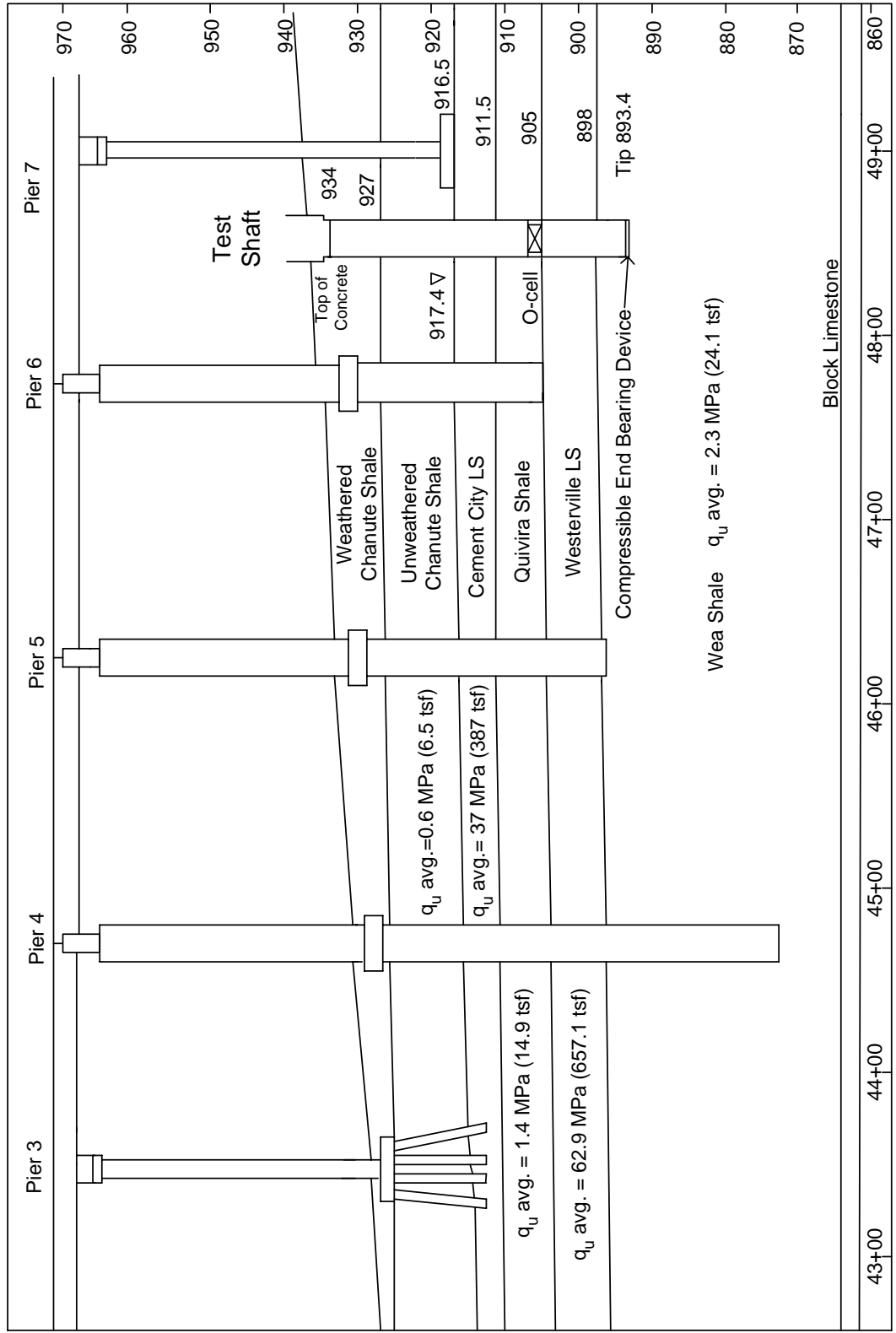
Figure 5.6 - Elevation view of Piers 3 thru 7 for bridge A6252.

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Wea Shale Member: The Wea Shale Member of the Cherryvale Formation is a bluish-gray, silty, micaceous shale. This member was encountered from about elevation 864.0 to 898.1 ft and averages about 34.1 feet (10.4 m) in thickness. Standard Penetration Test blow counts in the Wea Shale Formation averaged 100 blows in 12 inches (30 cm). Jar slake tests performed on this material produced jar slake indices from 3 to 6 as shown in Figure 5.7 and summarized in Table 5.1.
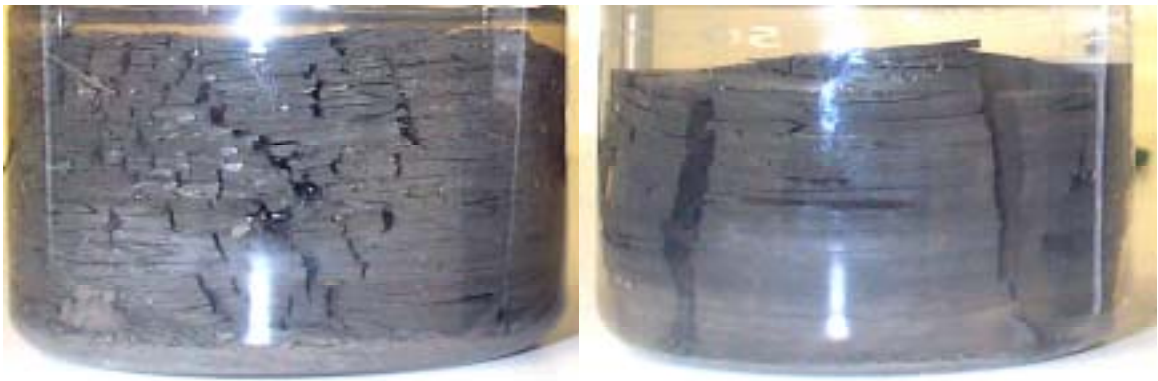
(a) Slake Index (3)   (b) Slake Index (6)

Figure 5.7- Range of jar slake index test results for Wea Shale Member: (a) elevation 986.0 ft, slake index (3), (b) elevation 880.8 ft, slake index (6).

Westerville Limestone Member: The Westerville Limestone Member consists of a lower, even-bedded limestone and an upper, oolitic limestone. This member was encountered from about elevation 898.1 to 904.8 ft. and averages about 6.7 feet (2.04 m) in thickness. RQD values for this stratum varied from 34 to 100 percent.

Quivira Shale Member: The Quivira Shale Member includes a gray shale in the lower and middle parts, a thin clay in the upper part, and a overlying slightly fissile dark gray shale. This member was encountered from about elevation 904.8 to 911.3 ft and averages about 6.5 feet (1.98 m) in thickness. Jar slake tests performed on this material produced jar slake indices of 1 to 4 as shown in Figure 5.8 and summarized in Table 5.1.

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a. Slake Index (1)   b. Slake Index (4)

Figure 5.8- Range of jar slake index test results for Quivira Shale Member: (a) elevation
905.7 ft., slake index (1), (b) elevation 910.0 ft., slake index (4).

5.2.2 Drum Limestone Formation

The Drum Limestone Formation contains two limestone members (from the base

upwards): the Cement City Limestone Member and the Corbin City Limestone Member.

In Missouri, only the lower limestone member (Cement City) has been recognized to

date. The Cement City Limestone Member is a gray to buff limestone. This member was

encountered from about elevation 911.3 to 916.5 ft and averages about 5.2 feet (1.58 m)

in thickness. RQD values varied from 22 to 100 percent.

5.2.3 Chanute Shale Formation

The Chanute Shale Formation consists of a silty, gray or maroon claystone in the

lower part, overlain by a silty to sandy shale. An un-weathered portion of this formation

was encountered from about elevation 916.5 to 927.2 ft. This portion averages about 10.7

feet (3.26 m) in thickness. Above elevation 927.2 ft to the base of the Raytown

Limestone Member of the Iola Formation, at about elevation 938.8 ft, the Chanute Shale

was weathered. Standard Penetration Test blow counts in the Chanute Shale Formation

averaged 100 blows in 8.8 inches (22 cm) above elevation 927.2 ft and 100 blows in 3

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inches (7.6 cm) below elevation 927.2 ft. Jar slake indices for this material varied from

1 to 4 as shown in Figure 5.9 and summarized in Table 5.1.

a.   Slake Index (1)   b.   Slake Index (4)

Figure 5.9- Range of jar slake index test results for Chanute Shale Member: (a) elevation
920.0 ft., slake index (1), (b) elevation 916.2 ft., slake index (4).

Table 5.1- Results of jar slake index tests of shale formations at the Grandview Triangle
site.

Formation	Elevation	2	5	10	30	1440
	(ft)	(min)	(min)	(min)	(min)	(min)
Chanute	920.0	2	2	2	1	1
Chanute	918.2	3	4	4	4	4
Quivira	909.6	4	4	4	4	4
Quivira	905.9	2	1	1	1	1
Wea	895.7	4	4	4	3	3
Wea	892.2	5	5	5	4	4
Wea	890.4	4	4	4	4	4
Wea	887.4	5	5	5	5	5
Wea	880.8	6	6	6	6	6
Wea	878.8	3	5	5	5	5
Wea	874	5	5	5	5	5
Wea	869.7	6	6	6	6	6

5.3 Unconfined Compressive Strength of Bedrock

The bedrock profile was divided into five layers based on strata. Average values

and ranges of the unconfined compressive strength for each layer are summarized in

Table 5.2. The bedrock was sampled by split spoon (SPT) and cored. The core was

logged with the amount of core recovered and the RQD being noted. Samples of the NX

core were returned to the laboratory for further testing. As shown in the table, the

unconfined compressive strengths of the limestones are significantly higher than the

shales and will take most of the axial load. The strength of the limestones exceeds the

strength of 4000 psi = 288 tsf (27.6 MPa) concrete and the strength of concrete would

control the ultimate side shear capacity. The strength of the shales ranges from 6.5 to

24.1 tsf (0.6 to 2.3 MPa) and generally increases with the depth of the stratum.

Table 5.2- Unconfined compressive strengths of rock strata at the Grandview Triangle site.

Strata	Elev.	Avg. q_u		Range	Std. Dev.
	ft.	MPa	tsf	tsf	tsf
Chanute	916.5 - 927.2	0.6	6.5	2.7 - 10.1	3.3
Cement City	911.3 - 916.5	37.0	386.6	118.6 - 588.0	179.8
Quivira	904.8 - 911.3	1.4	14.9	11.1 - 18.2	3.0
Westerville	898.1 - 904.8	62.9	657.1	164.8 - 1104	311.9
Wea	< 898.1	2.3	24.1	8.4 - 33.1	8.7

5.4 Foundation Design

Proposed bridge A6252 will cross US 71 at an angle as shown in Figure 5.10. Piers

4, 5, and 6 will each be supported by one column in the median of US 71. One column of

proposed bridge A6254, Pier 11, will also be located in the median. Construction of

spread footing or pile caps in the median of US 71 would likely require a detour to allow

room for the footing excavation including the use of sheet piling. Drilled shafts socketed

into bedrock would not require a detour and were therefore investigated as the preferred

foundation type.

Figure 5.10- Grandview Triangle load test site (from HNTB 2002).

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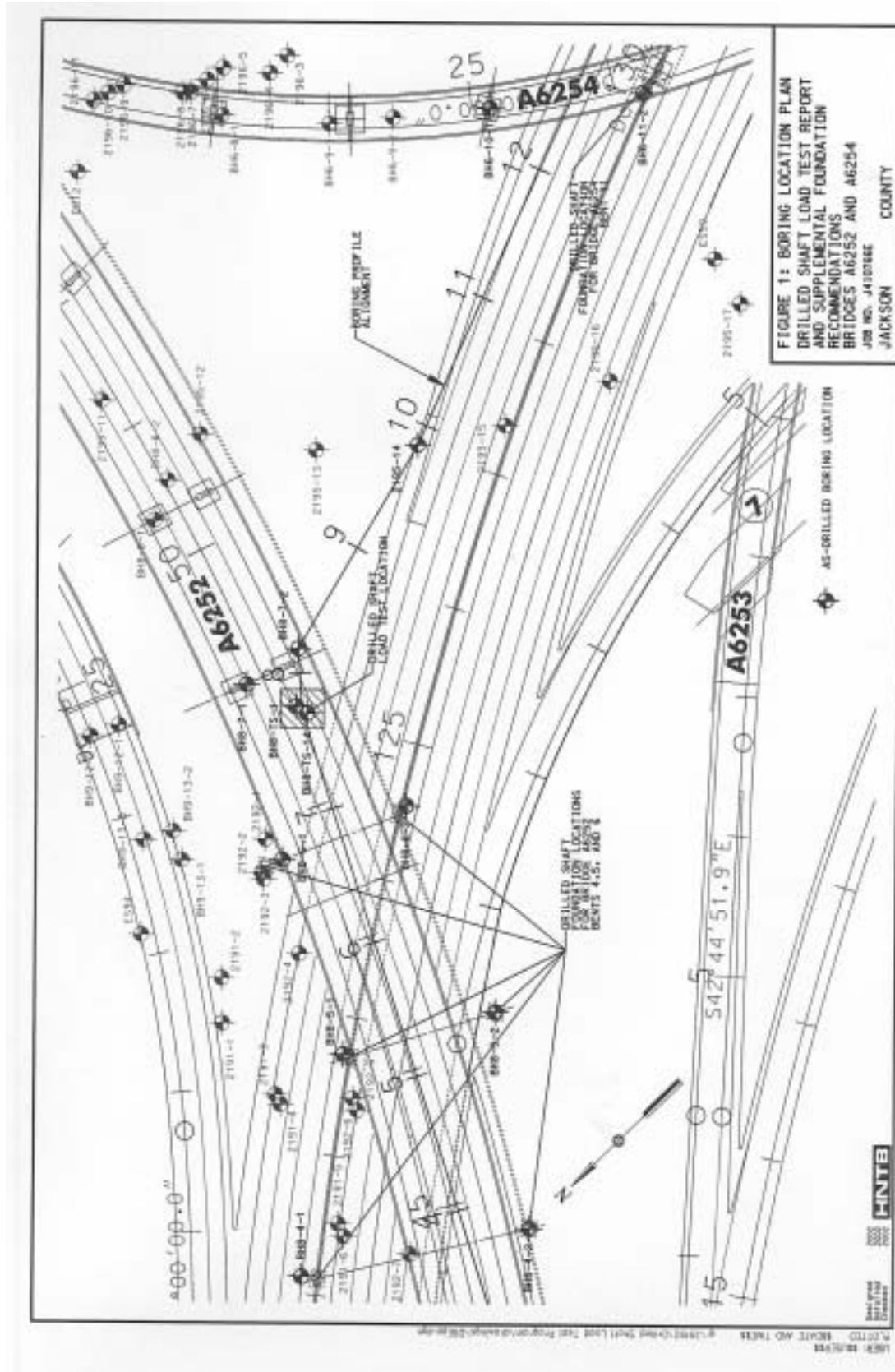
Since the closest limestone member (Winterset Limestone at about elevation 852 ft) thick enough to support drilled shafts and axial loads in end bearing is about 80 feet (24.4 m) below the top of the drilled shaft elevation, an Osterberg cell load test was recommended by HNTB (2002) to investigate side shear characteristics of the alternating layers of shale and limestone at shallower depths. The test shaft location was selected to be adjacent to Pier 6 on Bridge A6252 as shown in Figures 5.6 and 5.10.

5.5 Construction of Test Shaft

The test site was located on the east side of U.S. Highway 71 between proposed bridges A6252 and A6254 at Station 125+05, 90 ft. left of centerline. This site provided access for construction equipment and avoided existing roadways, utilities, and planned foundations. The site was also as close as possible to the planned production shafts.

A 34 inch (870 mm) diameter O-Cell^{TM} was selected to evaluate the side shear capacities of the different strata of shale and limestone as shown in Figure 5.11. To focus the test on the unit side shear capacities, a compressible inclusion was placed at the base of the rock socket as shown in Figure 5.12. The compressible end-bearing device, consisting of 3 inches (76 mm) of styrene foam sandwiched between two one half inch (12.5 mm) thick steel plates was placed at the base of the carrying frame and lowered into the shaft prior to concrete placement.

The test shaft was constructed by Clarkson Construction and their drilling subcontractor, Hayes Drilling, using a Watson 3100 drill rig shown in Figure 5.13. Work began on May 28, 2002 with drilling of the weathered limestone and shales at the surface with a double-flight 84 inch (2.1 m) bullet tooth rock auger. Temporary casing

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was seated into the Chanute shale. The remainder of the rock socket was excavated

alternating between a 72 inch (1.8 m) rock auger and core barrel. The last 2.5 foot long

(0.76 m) piece of Westerville Limestone core fell out of the core barrel while attempting

to remove it. The core fell back into the hole sideways.   Unsuccessful attempts where

made to remove the misaligned core with a couple of different sizes of core barrels and

the 72 inch (1.8 m) rock auger. At this point, the hole had filled to the top of the Cement

City Limestone with seepage water and had to be pumped dry before further attempts to

remove the core could proceed. The core was eventually broken up using a 36 inch (914

mm) core barrel and the pieces removed using the rock auger.   The excavation of the

rock socket was completed on May 31, 2002.

Water was observed entering the excavation from cracks in the Cement City

Limestone at a rate of 3 feet (0.9 m) per hour. The hole was pumped dry and both sonic

caliper and down-hole camera inspections were performed. The sonic caliper log is

shown in Figure 5.14. The camera was used with a tape to determine exact elevations of

the bedrock layers. After inspection, the hole was again pumped dry and the load frame

was lowered into the hole as shown in Figure 5.15. Figure 5.16 shows the placement of

the shaft concrete using a tremie line and pump truck on May 31, 2002. The slump of the

concrete varied from 7 inches (178 mm) for the first truck to 5.5 inches (140 mm) for the

last truck. The Osterberg cell load test was performed by Loadtest Inc. on June 3, 2002.
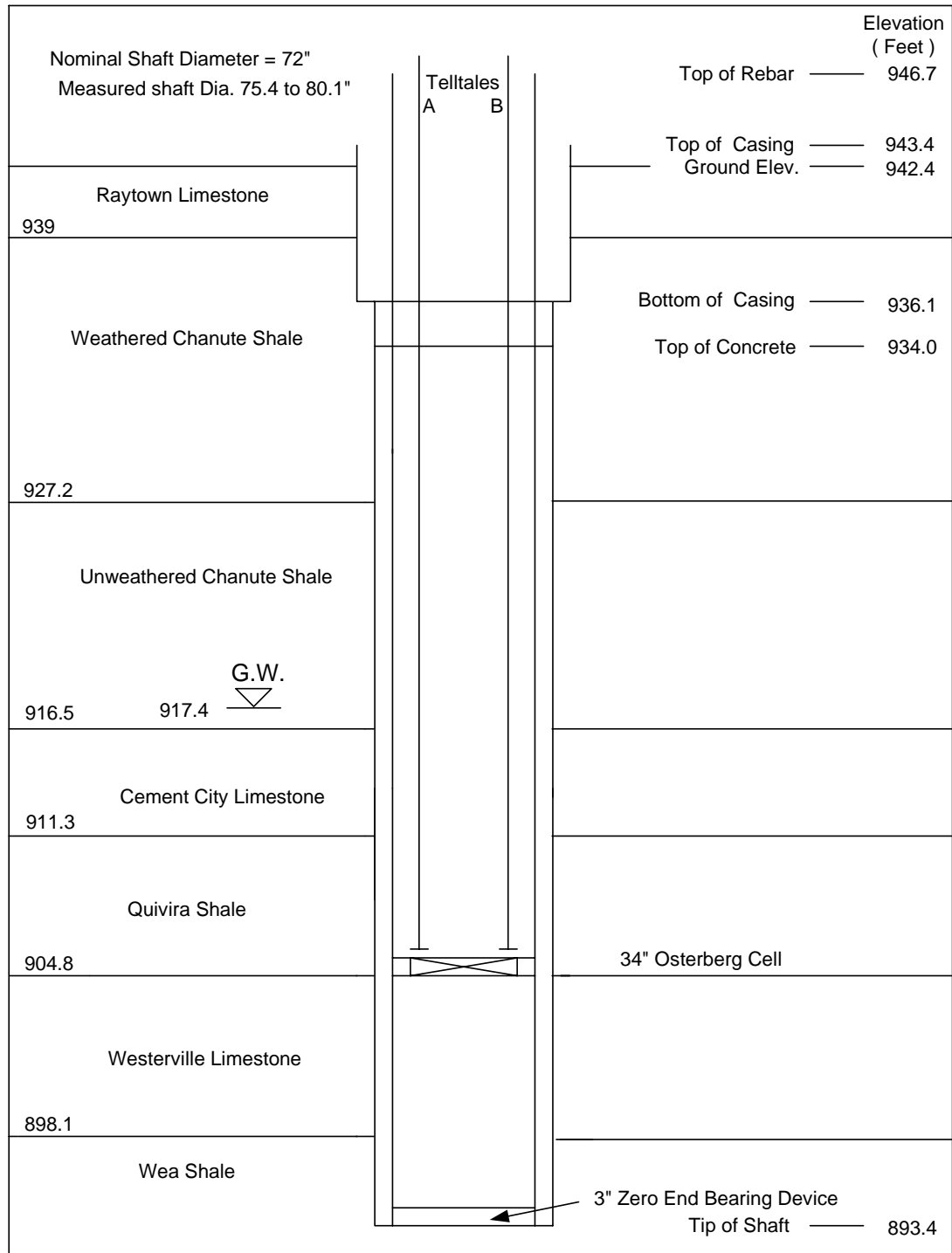
Figure 5.11- Schematic of Grandview Triangle test shaft and various shale strata.

Figure 5.12- Compressible end-bearing device.

Figure 5.13- Watson 3100 drill rig.
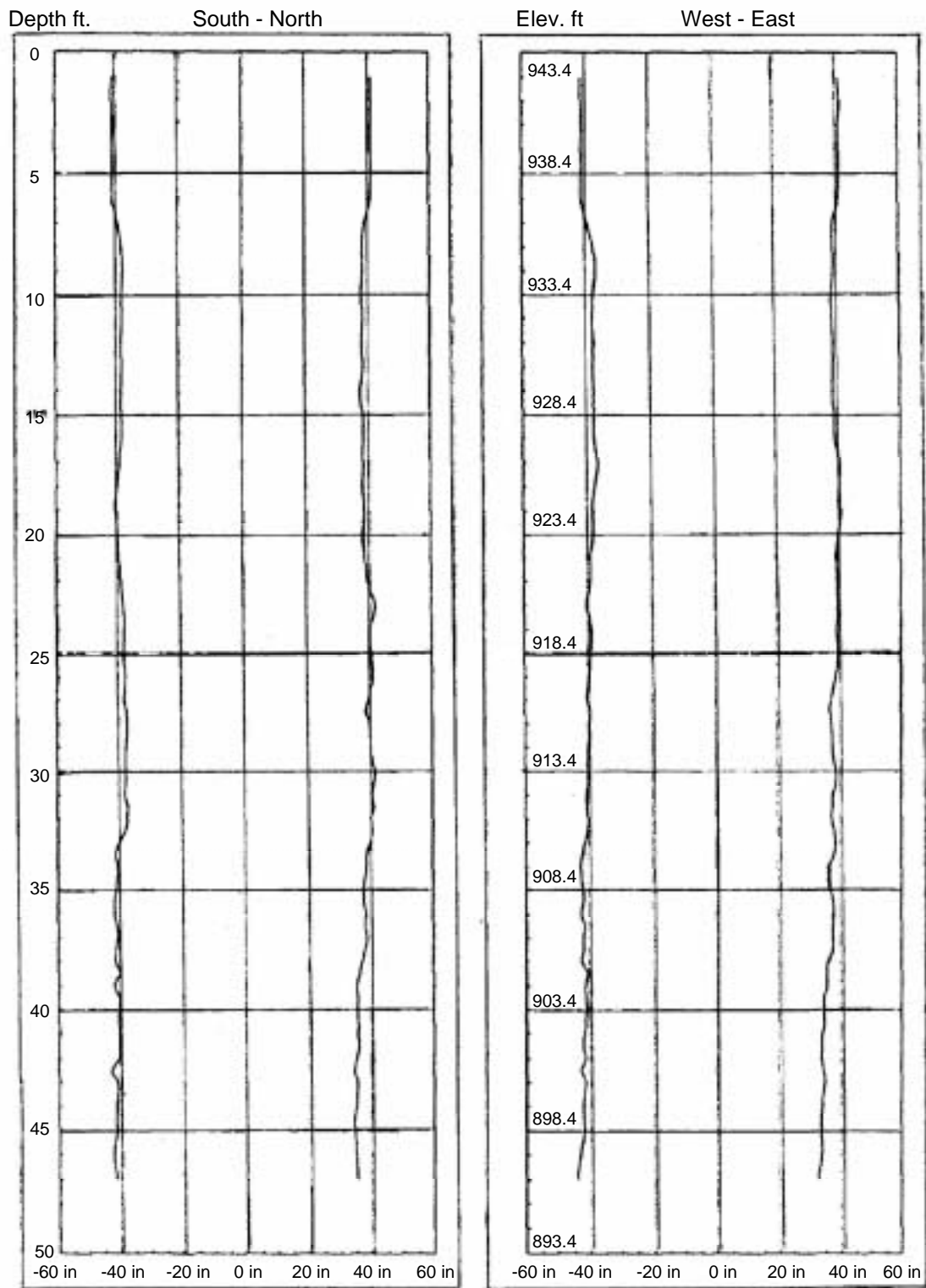
Figure 5.14- Sonic caliper log of Grandview Triangle test shaft (from HNTB 2002).

Figure 5.15- Lowering load frame into Grandview Triangle test shaft.

Figure 5.16- Placing shaft concrete using tremie and pump truck.

5.6 Load Test Setup and Procedures

The 34 inch (870 mm) diameter O-Cell^{TM}, with its base located 11.6 feet (3.54 m) above the tip of the rock socket, was pressurized to assess the combined end bearing and side shear below the O-Cell^{TM} and side shear above the cell. Initial assessments of the side shear capacities indicated that the segment of shaft below the O-Cell^{TM} would fail in side shear first. After determination of the side shear in the lower segment of the shaft, continued loading would completely compress the end-bearing device and begin mobilizing end-bearing. The combined end-bearing and side shear below the O-Cell^{TM}

would then be used as a reaction to assess the side shear above the O-Cell^{TM}.

The O-Cell^{TM} was pressurized in 21 equal increments of 600 psi (4,137 kPa) to a maximum O-cell^{TM} pressure of 12,610 psi (86.9 MPa), which corresponds to a load of 3,856 tons (34.3 MN) in each direction. The loading increments are denoted as 1L-1, 1L-2, and 1L-3, etc.; the unloading events are denoted as 1U-1, 1U-2, 1U-3, etc. At the maximum load of 3,856 tons (34.3 MN) the upper segment of the shaft was displacing rapidly and higher loads could not be achieved. The O-Cell^{TM} was then unloaded in 5 equal increments and the test was concluded. The applied load increments followed procedures in ASTM D1143- Standard Test Method for Piles Under Static Axial Compressive Load.

Test shaft instrumentation is shown in Figure 5.17 and a summary of dimensions is given in Table B.1 in Appendix B. Expansion of the O-Cell^{TM} was measured by three LVWDTs (Geokon Model 4450 Series) positioned between the lower and upper plates of the O-Cell^{TM}. Compression of the shaft between the O-Cell^{TM} and the compressible end-bearing device at the base of the shaft was measured by a pair of embedded compression telltales (ECTs). Telltale casings were attached to the carrying frame and the upper plate of the O-Cell^{TM} to monitor top of shaft movement and top of cell movement, respectively.

Strain gages were used to assess the load transfer in side shear of the shaft above and below the O-Cell^{TM}. Six levels of two sister bar vibrating wire strain gages were installed in the shaft - four levels above the cell and two below. Final positioning of the strain gages was determined by the down-hole camera, which was used to determine exact elevations of the bedrock layer interfaces.

| Nominal Shaft Diameter = 72" | | | | Elevation (Feet) |
| Measured shaft Dia. 75.4 to 80.1" | | | Top of Rebar ——— | 946.7 |

Telltales A B

Raytown Limestone
939

Top of Casing ——— 943.4
Ground Elev. ——— 942.4

Weathered Chanute Shale

Bottom of Casing ——— 936.1
Top of Concrete ——— 934.0

927.2

SG6 23282 & 23283 ——— 927.0

Unweathered Chanute Shale

SG5 23280 & 23281 ——— 921.0

G.W.
916.5   917.4 ▽

SG4 23278 &23789 ——— 916.5

Cement City Limestone
911.3

SG3 23276 &23277 ——— 911.5

Quivira Shale
904.8

LVWDT 66176, 177, 178
34" Osterberg Cell (2173-3) ——— 905.0

Westerville Limestone

SG2 23274 &23275 ——— 901.0
ECT 66171 & 66172
898.1

SG1 23272 &23273 ——— 898.0

Wea Shale

| = strain gages

3" Zero End Bearing Device
Tip of Shaft ——— 893.4

Figure 5.17- Schematic of Grandview test shaft instrumentation.

5.7 General Test Results

Results of the Osterberg cell load test at the Grandview Triangle site are presented in this section. Detailed load test data, including a summary of test shaft dimensions, load-displacement data, strain gage data, and computed unit side shear data is presented in Appendix B.

Upward and downward load displacement curves determined from the load test at the Grandview Triangle test site are shown in Figure 5.18. The maximum gross load applied to the base of the shaft occurred at load interval 1L-21 and equaled 3,856 tons (34.3 MN) in each direction. At this loading, the O-Cell^{TM} had expanded 1.63 inches (41.45 mm) with 1.41 inches (35.8 mm) of upward movement and 0.22 inches (5.67 mm) of downward movement. At the maximum load, the ultimate capacity of the rock socket above the O-Cell^{TM} was reached and additional load could not be applied to the rock socket below the O-Cell^{TM}.

The maximum net load applied to the upper portion of the shaft was equal to 3,856 tons (34.30 MN) minus the buoyant weight of the shaft, 59 tons (0.53 MN), for a net applied load of 3,797 tons (33.78 MN). The maximum net load applied to the shaft segment below the O-Cell^{TM} was equal to 3,856 tons (34.30 MN). Due to the minimal downward displacement and the use of the compressible end-bearing device, the entire load below the cell was assumed to be carried by side shear only.
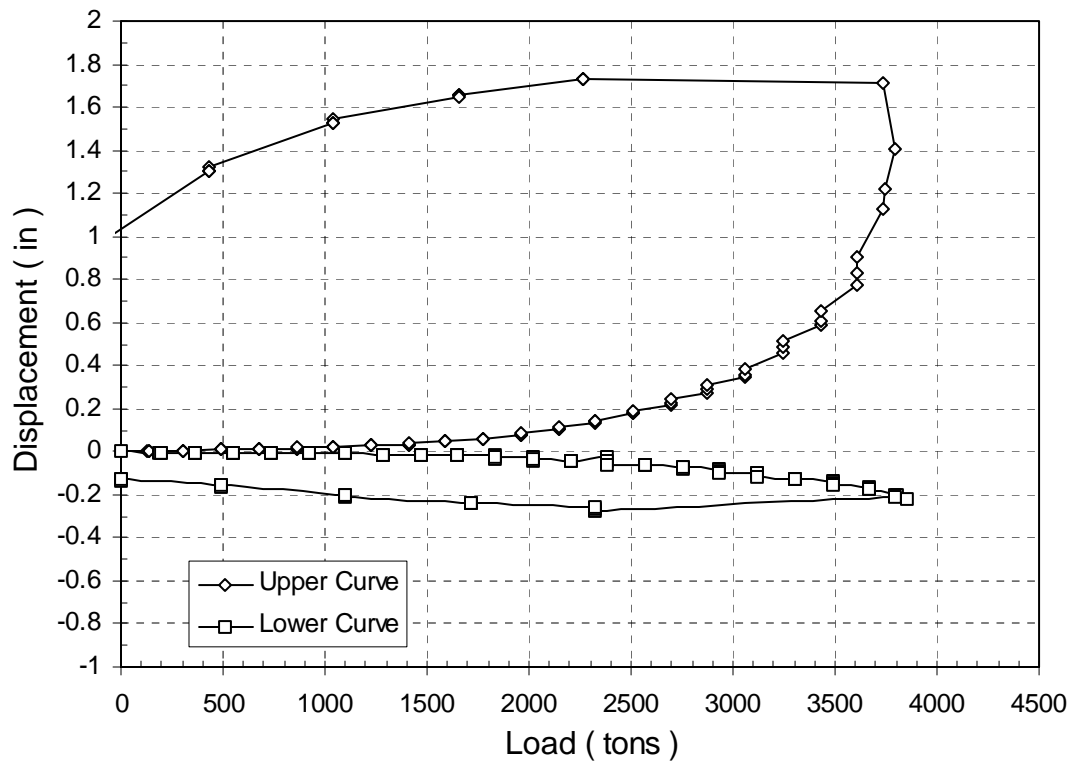
Figure 5.18- Measured load-displacement curves for upward and downward loading of test shaft at the Grandview Triangle site.

The equivalent top-down load-displacement curves are shown in Figure 5.19. None of the load was transferred in end bearing and the equivalent top-down load-displacement curve does not include end bearing. The equivalent top-down load-displacement curve adjusted for additional elastic compression indicates that a shaft loaded from the top with a load of 5,463 tons (48.6 MN) would settle about 0.25 inches (6.4 mm), of which 0.15 inches (3.9 mm) is estimated to be from elastic compression of the shaft.

Equivalent Top Load (tons)
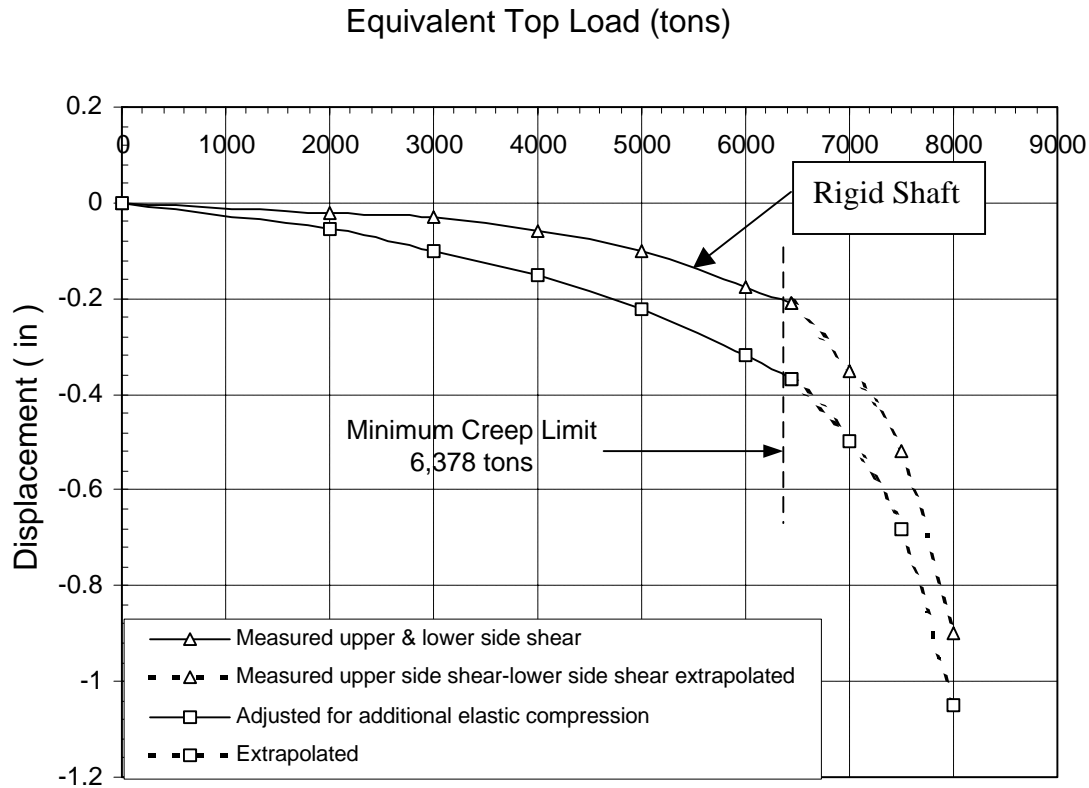
Figure 5.19- Equivalent top-down load-displacement curves for the Grandview Triangle test shaft.

The distribution of axial force with depth or elevation at various load increments is generated from strain gage data, the equivalent modulus of the shaft, and the cross-sectional area of the shaft as described in Chapter 3. The distribution of axial force with elevation for the Grandview Triangle test is shown in Figure 5.20 for different loading levels. Detailed strain gage data used to compute these forces is given in Appendix B.

On the day of the test, the concrete compressive strength, as determined from concrete test cylinders, was 6000 psi (41.37 MPa). Equation 3.3 was used to calculate an elastic modulus for the concrete of 4,415,000 psi (30,230 MPa). This, combined with the area of reinforcing steel and shaft diameters- 77.8 inches (1976 mm) above the O-Cell^{TM} and 76.3 inches (1938 mm) below- was used to determine an average shaft stiffness (AE) of 21,000,000 kips (93,853 MN).

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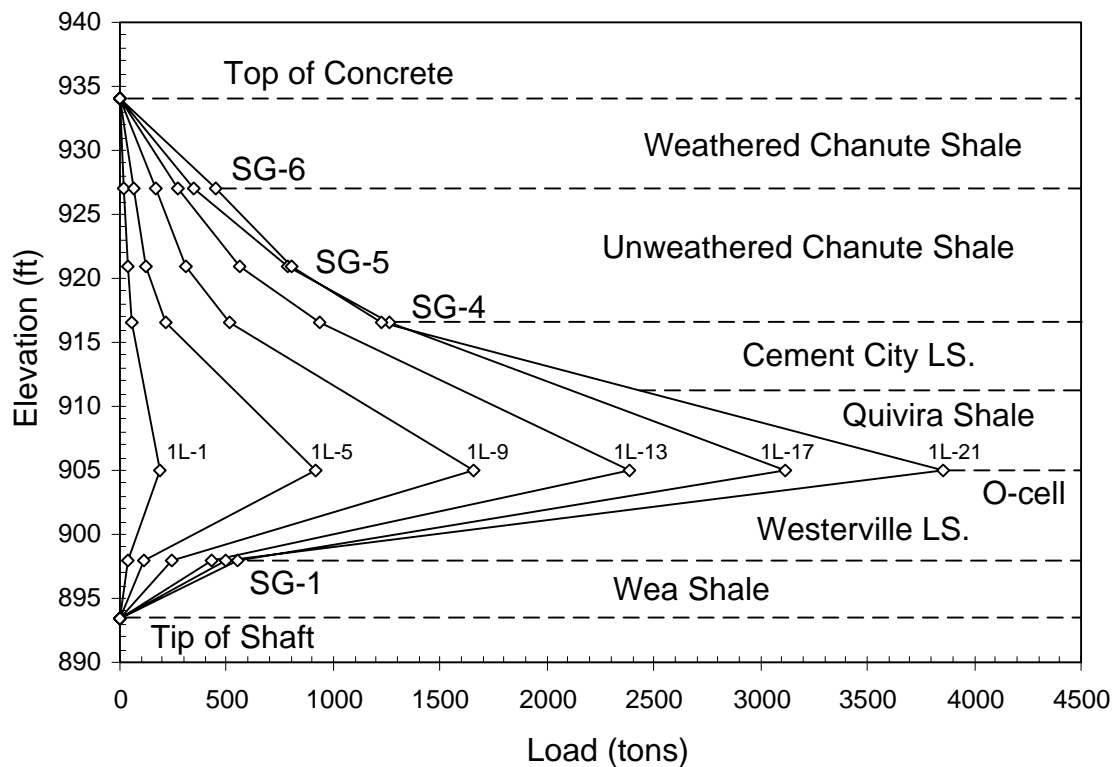
Figure 5.20- Distribution of axial force for the Grandview Triangle test shaft.

Figure 5.21 shows the mobilized unit side shear plotted versus O-cell[TM]

displacements for the different strata encountered at the Grandview test site. As shown in

Figure 5.21, the ultimate unit side shear was mobilized in the weathered and unweathered

Chanute Shale strata and the Wea Shale strata. No clear peak in the unit side shear was

observed in the Quivira Shale and Cement City strata or in the Westerville Limestone

stratum. The maximum mobilized unit side-shear values for the shaft based on strain gage

data are shown in Table 5.3 and plotted in Figure 5.22. The ultimate unit side shear of

the unweathered Chanute Shale varied from 3.1 to 4.8 tsf (296.9 to 407.6 kPa). The

weathered Chanute Shale had an ultimate side shear of 3.2 tsf (301 kPa). The average

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ultimate side shear for the entire thickness of Chanute Shale was 3.6 tsf (346.8 kPa). The

ultimate unit side shear for the combined Cement City Limestone and Quivira Shale was

10.9 tsf (1039.4 kPa). The Wea Shale had an ultimate side shear value of 6.4 tsf (612.9

kPa). The Westerville Limestone did not achieve an ultimate side shear during this load

test. A maximum unit side shear of 24 tsf (2,298 kPa) was achieved for the Westerville

Limestone before the Osterberg cell load test was concluded.
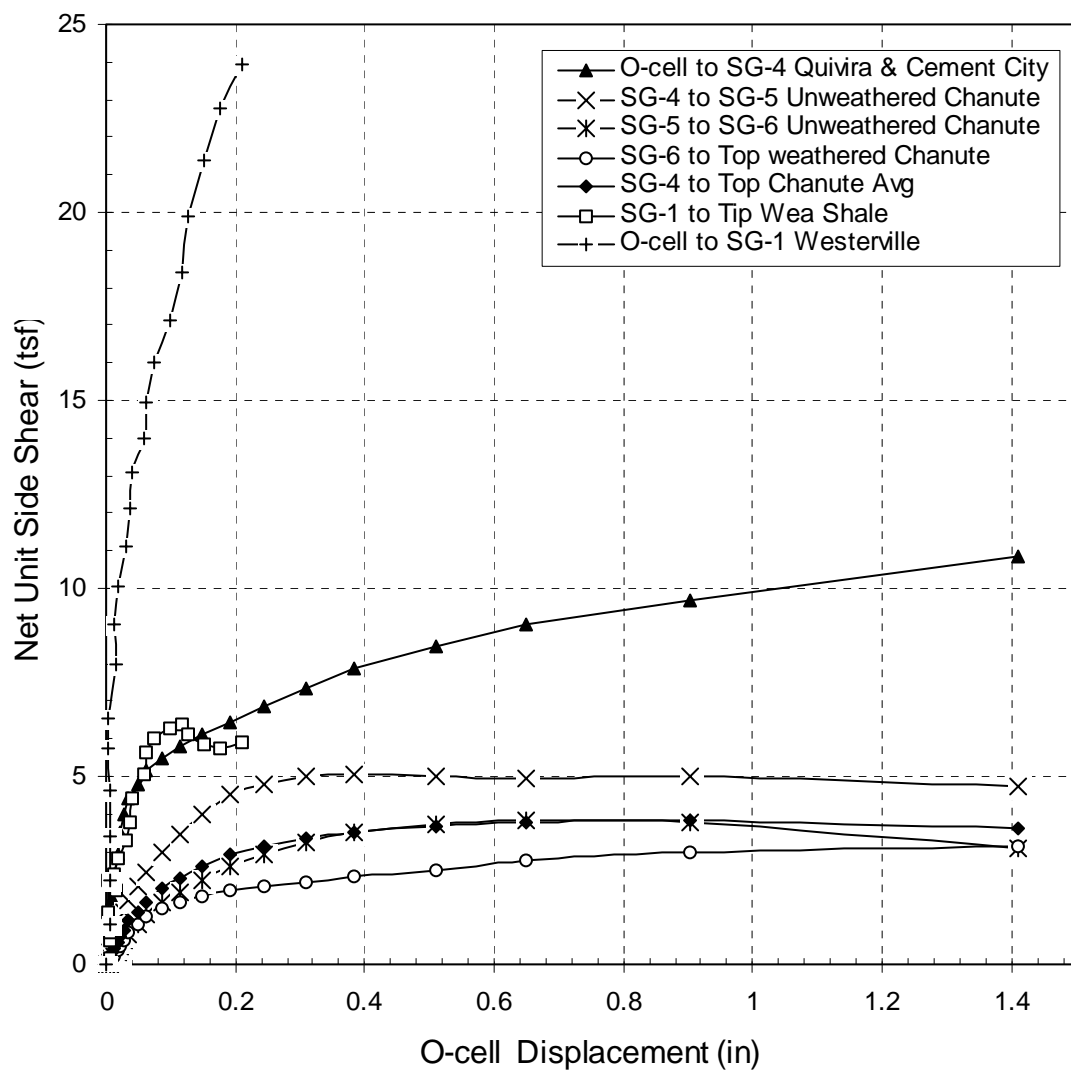
5.21- Mobilized unit side shear versus O-cell displacement for various geologic strata at
the Grandview Triangle test site.

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Level 2 and level 3 strain gages were too close to the O-Cell^{TM} and did not produce

reliable data. Because the steel plates above and below the O-cell^{TM} are not infinitely

stiff, a cone of compression is believed to develop above and below the O-cell^{TM}, as

shown in Figure 5.23, that distributes the applied load out to the sides of the socket. The

level 2 and 3 strain gages where located near the sides of the rock socket, in a transition

zone where material at the sides of the socket is believed to be in tension.   Since the level

3 strain gages did not function properly, unit side shear values for the Cement City

Limestone and Quivira Shale could not be determined directly from the strain gage data.

A value of unit side shear of 4.8 tsf (460 kPa) was therefore assumed for the Quivira

Shale in order to calculate the unit side shear of the Cement City Limestone. The

assumed unit side shear value was determined by a comparison of calculated unit shear

values and unconfined compressive strength data for all shale formations at the site.

Table 5.3-Unit side shear values calculated from strain gage data for the Grandview Triangle test shaft.

Load Transfer Zone	Strata	Elevation (ft)	Unit Side Shear	
			kPa	tsf
Top of Shaft to SG-6	Weathered Chanute	934 - 927	301	3.2
SG-6 to SG-5	Chanute	927 - 921	295	3.1
SG-5 to SG-4	Chanute	921 - 916.5	453	4.8
SG-6 to SG-4	Chanute	927 - 916.5	363	3.8
SG-4 to O-Cell^{TM}	Combined Quivira & Cement City	916.5 - 905.0	>1,039	>10.9
SG-4 to SG-3	Cement City	916.5 - 911.5	>1,651*	>17.2*
SG-3 to SG-2	Quivira	911.5 - 905.0	460*	4.8*
O-Cell^{TM} to SG-1	Westerville	905.0 - 898.0	>2,293	>24.0
SG-1 to Tip of Shaft	Wea	898.0 - 893.4	565	5.9

* The value for the Quivira was assumed in order to calculate the unit side shear of the Cement City Limestone.
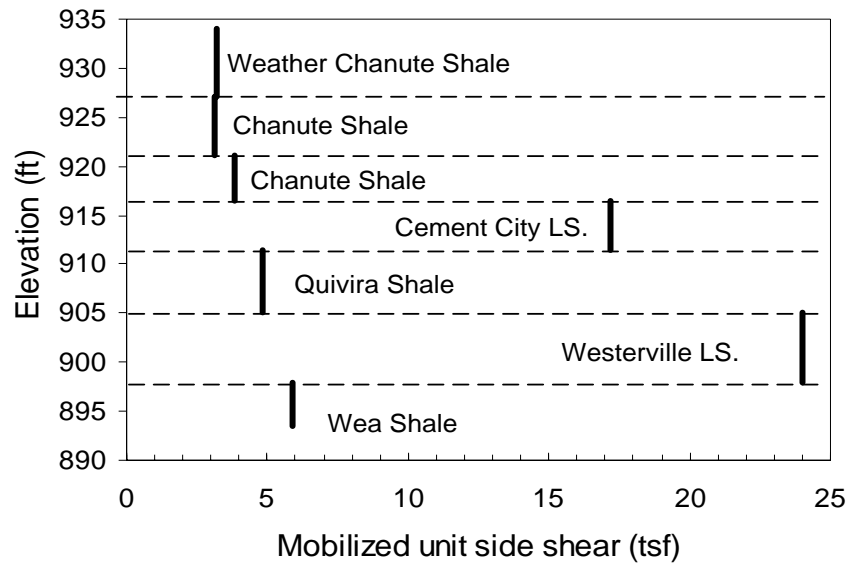
Figure 5.22- Mobilized unit side shear calculated from strain gage data for Grandview Triangle test shaft.
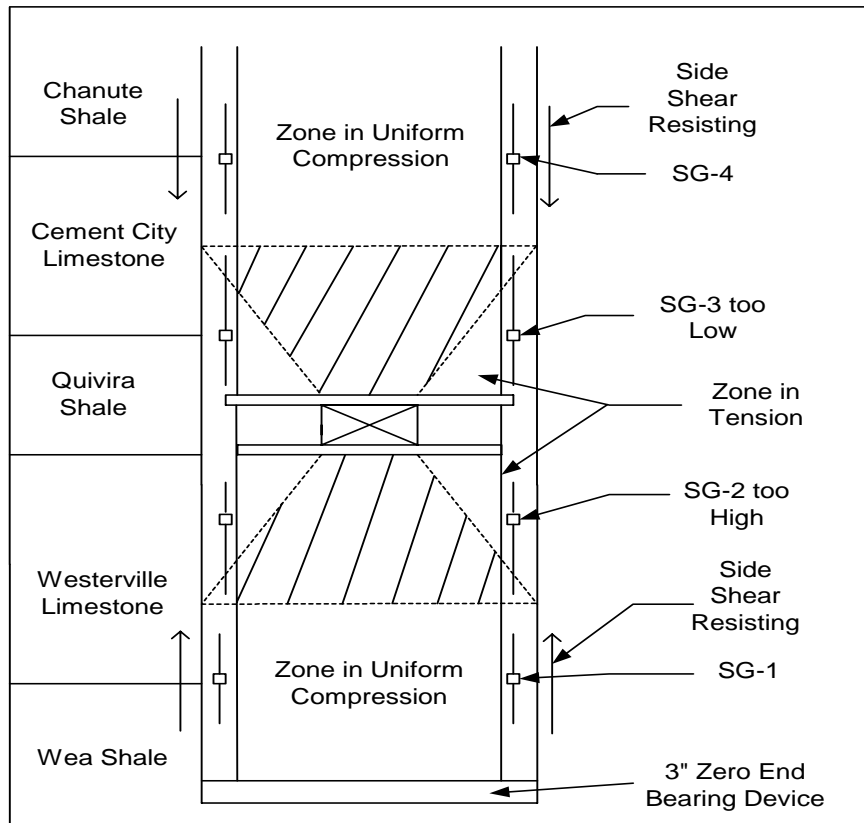
Figure 5.23- Zone of influence for level 2 and 3 strain gages.

As described in Chapter 3, the creep limit is determined by plotting the additional displacement that occurs over the time interval 2 to 4 minutes after application of the load while the load is maintained as shown in Figures 5.24 and 5.25. A creep limit of 2,600 tons (23.13 MN) was reached for the upper portion of the shaft at a displacement of 0.217 inches (5.51 mm). At a maximum loading of 3,856 tons (34.3 MN), no apparent creep limit was reached for the lower portion of the shaft with a displacement of 0.223 inches (5.67 mm). Based on these results, Loadtest recommended that significant creep would not occur for a top loaded shaft until a load greater than 6,378 tons (56.73 MN) is exceeded by some unknown amount.
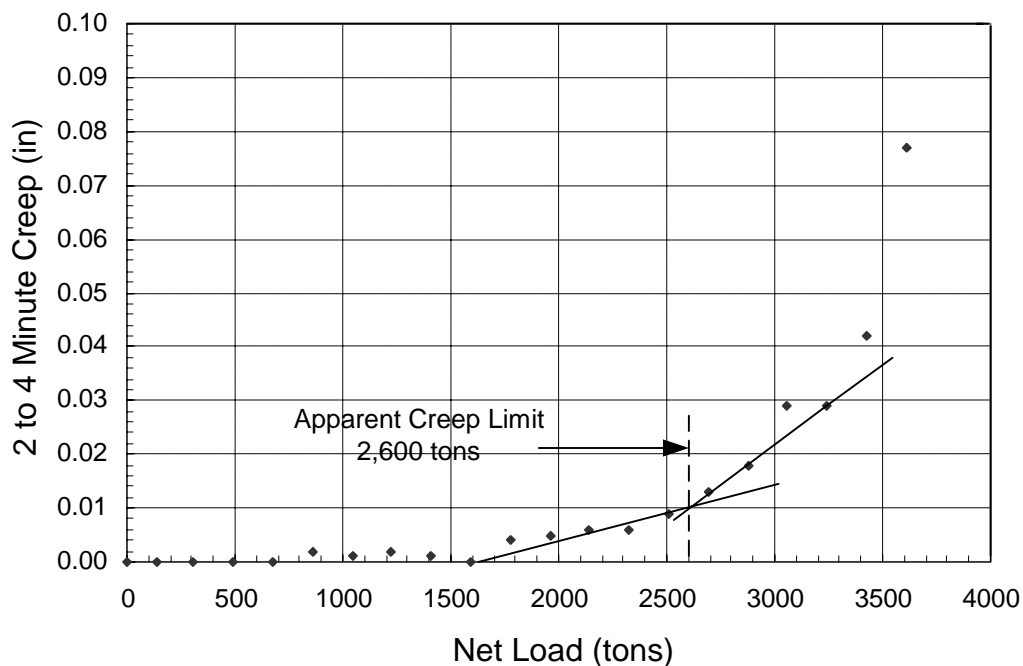
Figure 5.24- Creep displacement for the upper portion of Grandview Triangle test shaft.
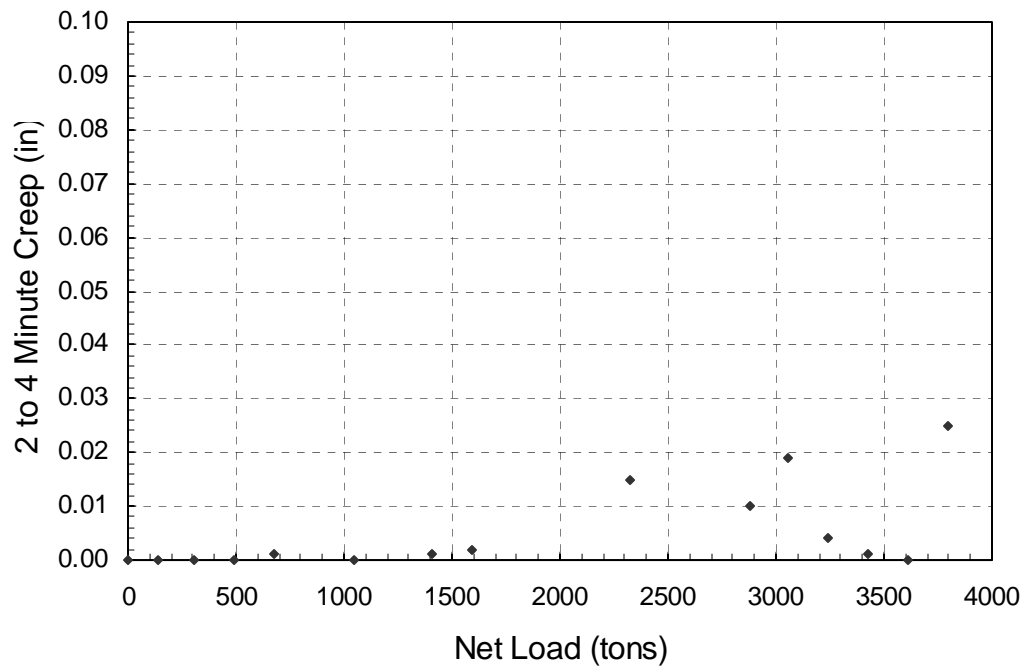
Figure 5.25- Creep displacement for the lower portion of Grandview Triangle test shaft.

5.8 Practical Applications

Data from the Osterberg cell load test would allow the 7.5 feet (2.3 m) diameter rock sockets at bridge A6252 to be shortened a total of 214 feet (65.2 m) for a cost savings of $214,000. The cost of the test shaft excavation and the Osterberg cell load test was $195,000 for a net savings of $19,000. Additional cost savings of $50,000 dollars were anticipated for Pier 11 of bridge A6254 but the footing was subsequently designed as a spread footing. Although the net cost savings of $19,000 was not significant, it indicates the magnitude of cost savings that could be realized in applying Osterberg cell load test information to future projects in this area.

5.9 Discussion

Several points regarding the interpretation of results from the O-cell load tests warrant discussion. Although the upper segment of the rock socket mobilized the full side shear capacity, the ultimate unit side shear for the Cement City Limestone and the Quivira Shale could not be determined individually since the strain gages at the Cement City-Quivira contact did not function properly. The level 3 strain gages were too close to the O-Cell^{TM} and did not produce reliable data. Because the steel plates above and below the O-cell^{TM} are not infinitely stiff, a cone of compression develops above and below the O-cell^{TM} that distributes the applied load out to the sides of the socket. This is shown graphically for the zone above the O-cell^{TM} in Figure 5.26. The level 3 strain gages where located near the sides of the rock socket, in a transition zone where material at the sides of the socket is in tension. A possible solution would be to position the strain gages towards the center of the rock socket. This, however, would position the strain gages in a cone of influence of an unknown diameter. This may also be a point of error for the Osterberg test in general.

Since the level 3 strain gages did not function properly the unit side shear values could not be determined directly from the strain gage data for either the Cement City Limestone or Quivira Shale. A value of unit side shear of 4.8 tsf (460 kPa) was assumed for the Quivira Shale in order to calculate the unit side shear capacity of the Cement City Limestone. The revised distribution of axial force is shown in Figure 5.27. The assumed unit side shear value was determined by a comparison of calculated unit shear values and unconfined compressive strength data for the shale formations. The adjusted axial load curve indicates that only about 729.1 tons (6.5 MN) of the axial load is shed in the

Quivira Shale with most of the axial load 1,896.9 tons (16.9 MN) is carried by the

Cement City Limestone even though the Cement City Limestone is a thinner layer.
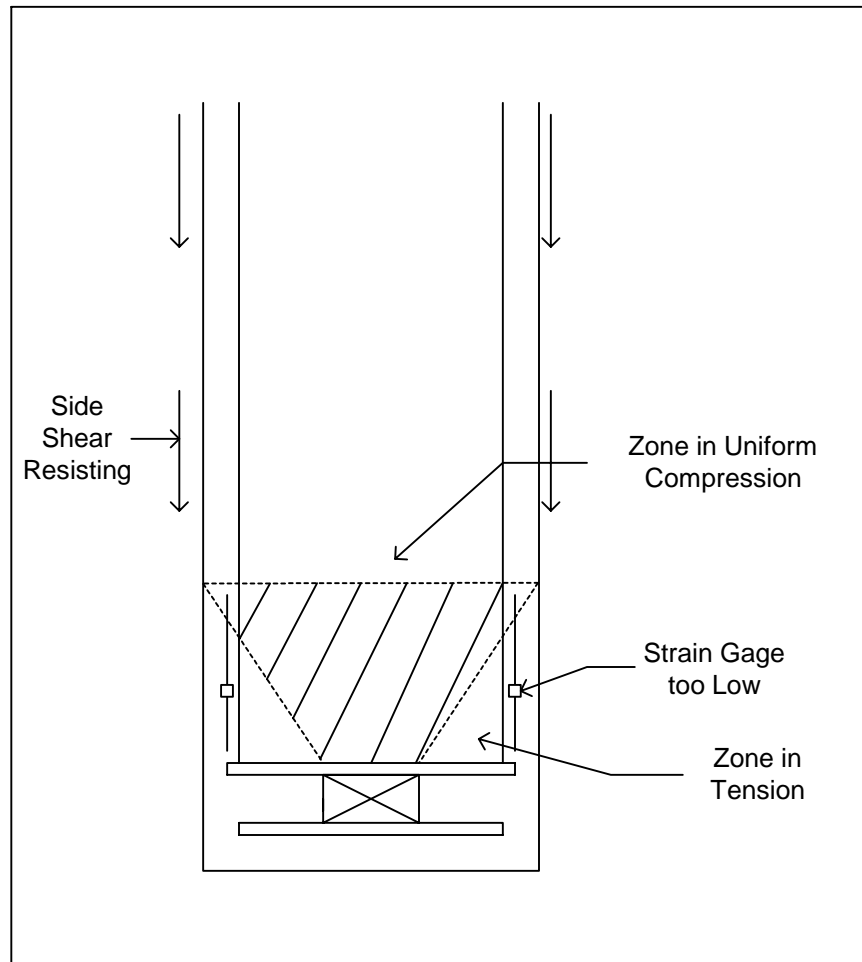
Figure 5.26- Influence of strain gage positioning (after Hayes and Simmonds 2002).
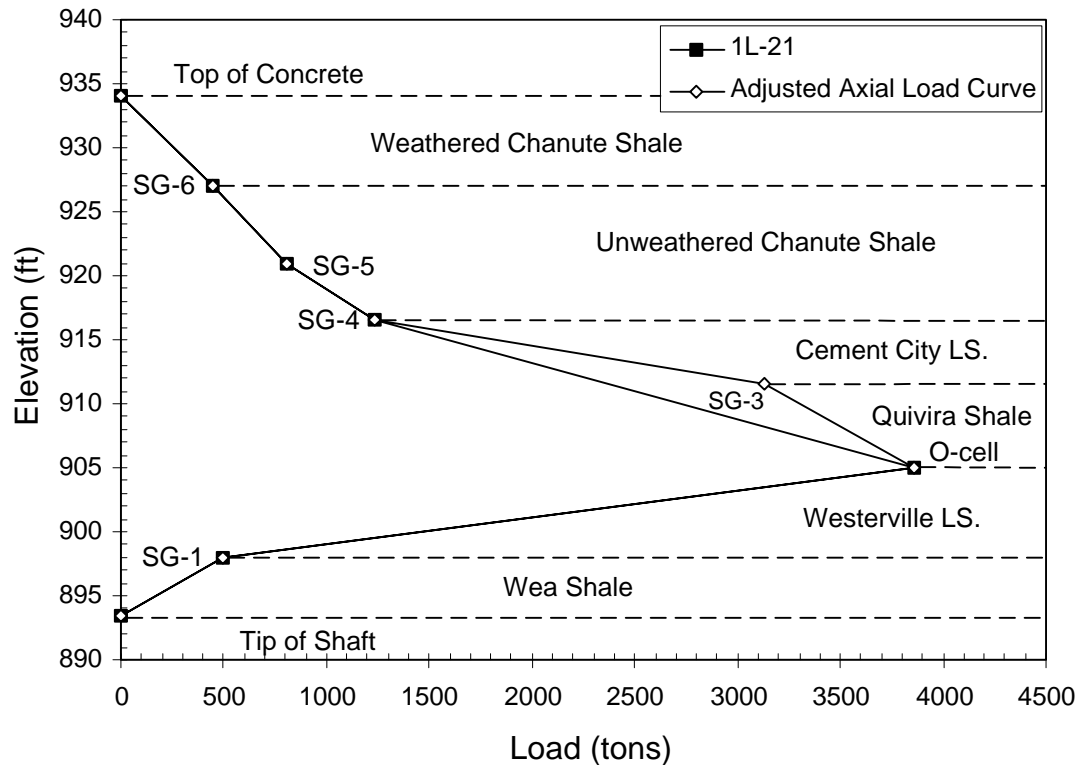
Figure 5.27- Adjusted axial load curve based on assumed unit side shear value for Quivira Shale.

Another issue with interpretation of Osterberg cell load tests involves when to subtract the buoyant weight of the shaft from the gross load to calculate the net load. Theoretically, the O-Cell™ does not impose an additional upward load until the O-cell™ force exceeds the buoyant weight of the shaft above the cell. Loadtest (2002) therefore uses the net load, defined as the gross O-Cell™ load minus the buoyant weight of the shaft above the cell, to determine side shear above the cell. The difference in the Grandview Triangle load test and the test at the Lexington site is that the buoyant weight of the shaft was not subtracted from the gross O-Cell™ load to calculate the lower side shear and end-bearing for the Grandview Triangle test shaft. However, Loadtest (2002) reports that the top-down load-displacement curve determined for the Grandview Triangle test shaft is still the same as the top-down load-displacement curves determined

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for Lexington test shafts because they added the net upward load and gross downward load and then subtracted the buoyant weight of the shaft.

5.10 Summary and Conclusions

The Missouri Department of Transportation (MoDOT) is currently reconstructing and updating an intersection in the Kansas City metropolitan area known as the Grandview Triangle. Three major interstate routes and 7 local routes intersect at this location. All the bridges, ramps, and pavement will be replaced with 25 new bridges and more than 40 retaining walls.

A twelve span structure (A6252) and a thirteen span structure (A6254) will cross route US 71 and will require the construction of footings in the median of US 71. The use of conventional spread footings or pile caps would require a detour to allow room for the footing excavation. Drilled shafts socketed into bedrock would not require a detour and where investigated as the preferred foundation type. Since the closest limestone layer thick enough to support the axial loads in end bearing only is about 80 feet (24.4 m) below the top of the shaft, an Osterberg cell load test was performed to investigate the unit side shear characteristics of the bedrock.

A 34 inch (870 mm) diameter O-Cell^{TM}, with its base located 11.6 feet (3.54 m) above the tip of the rock socket was pressurized to assess the combined end bearing and side shear below the O-Cell^{TM} and side shear above the cell. Initial assessments of the side shear capacities based on FHWA guide lines indicated that the segment of shaft below the O-Cell^{TM} would fail in side shear first. After determination of the side shear in the lower segment of the shaft, continued loading would compress the end-bearing device mobilizing the end-bearing. The combined end-bearing and side shear below the O-

Cell^{TM} would then be used as a reaction to assess the side shear above the O-Cell^{TM}. However, the combined side shear capacity of the Westerville Limestone and Wea Shale below the O-Cell^{TM} was greater than anticipated and the ultimate side shear capacity of the segment of the shaft below the O-cell^{TM} could not be determined. Although the side shear in the socket segment below the O-cell^{TM} was not fully mobilized, values achieved were greater than anticipated.

CHAPTER SIX
WAVERLY, MO. TEST SITE

A new bridge is proposed across the Missouri River in central Missouri on Route 65 in Waverly, Missouri. The bridge site is located about 18 miles downstream of the Lexington site. Foundation elements chosen for the piers in the river are to be drilled shafts socketed into bedrock. The bedrock at the bridge site is similar to the Lexington site and consists of older Pennsylvanian Age shales, siltstones, sandstones, limestones, and scattered coal beds. Present design methods used by MoDOT would dictate that as a result, rock sockets with a diameter of 6.5 ft. (1.98 m) would need to be as long as 63 feet (19.2 meters) in order to carry the anticipated axial load of 1525 tons (13.6 MN). The bridge design consultant, Harrington and Cortelyou, Inc. wanted to use both side shear and end bearing for the design of shafts at Pier 11. Due to the questionable nature of the bedrock at Pier 11 and the desire to use some end bearing, it was decided to test a rock socketed drilled shaft using an Osterberg load cell. Since time did not allow a load test to be conducted during the design phase of this project, the load test was conducted on a "production" drilled shaft at Pier 11. A 26 inch (660 mm) Osterberg load cell with a capacity of 1800 tons (16MN) in each direction was chosen to test the production shaft to twice the design load. The "production" test shaft was constructed by Jensen Construction Co. and the Osterberg cell load test was performed by Loadtest Inc. on September 30, 2002. The test indicated that the bedrock would be adequate to support design loads at Pier 11 using side shear and end bearing.

A general geologic description of the Waverly test site is presented in this chapter followed by a summary of the engineering characteristics of the most pertinent strata.

The construction and testing procedures for the test shaft are then described, followed by presentation of the results from the load test.

6.1 Site Description

The Missouri Department of Transportation (MoDOT) is planning a realignment of Route 65 at Waverly, Missouri. The realignment includes a new bridge across the Missouri River downstream of the present structure, which was opened to traffic in the 1920's.

The project is situated in the Missouri River alluvial plain in the central part of the state of Missouri as shown in Figures 6.1 and 6.2. The project was designed and constructed using English units; English units are therefore used in this chapter. The alluvial plain is mostly flat with some earthen levees constructed to protect row crop production. Currently the river channel is located adjacent to the rolling hills on the southern limit of the plain. The Missouri River alluvial plain is about 3600 feet (1.1 kilometers) wide in the project area and alluvial materials consist of 9 to 12 feet (2.7 to 3.7 meters) of cohesive soil overlying sand with scattered gravel layers. The thickness of the alluvial materials in the flood plain north of the river varies from 50 to 67 feet (15.2 to 20.4 meters). Two possible shipwrecks are thought to be in this area: the Tropic sunk in 1857 and the Grace Houston sunk in 1881. Within the river, the alluvial sand and gravel layers range in thickness from 25.7 to 45.8 feet (7.8 to 14 meters). Three further shipwrecks were noted in the channel of the Missouri River, but due to present dredging of the river for sand, it is highly unlikely that any remains will be found. The depth of wind blown loess on the rolling hills south of the river varies in thickness from 15 to 26 feet (4.6 to 7.9 meters). The former weigh station for the Steamboat Coal and Mining

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Company is located in the vicinity of Pier 13. A 1912 Bureau of Mines report, stated that

the Steamboat Coal and Mining Company operated a mine about 0.25 miles (0.4

kilometers) east of Waverly and the mine was 102 feet (31m) deep. In the vicinity of the

mine, the Waverly coal is 3 to 4 feet (0.9 to 1.2 m) thick. Borings at Piers 12 and 14

were advanced to an elevation sufficient to intercept the Waverly coal bed. No voids

were encountered at either pier and the mine is probably located farther to the east and

does not appear to be a concern for the proposed bridge.
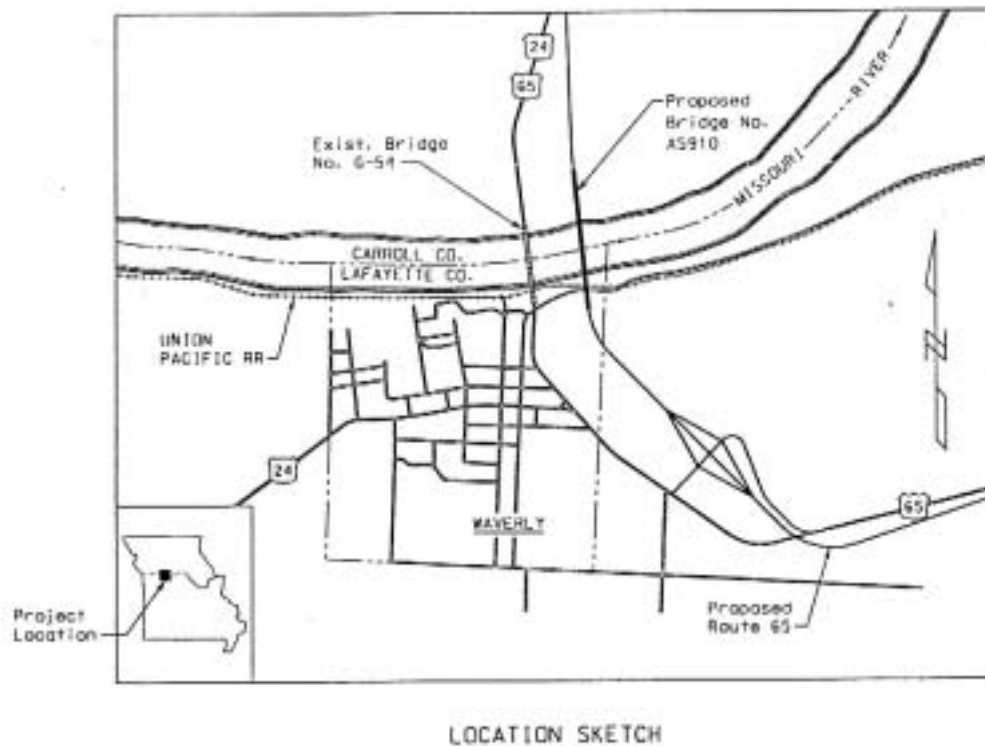
LOCATION SKETCH

Figure 6.1- Location sketch of Waverly bridge site.

Figure 6.2 Plan and elevation view of Waverly bridge site.

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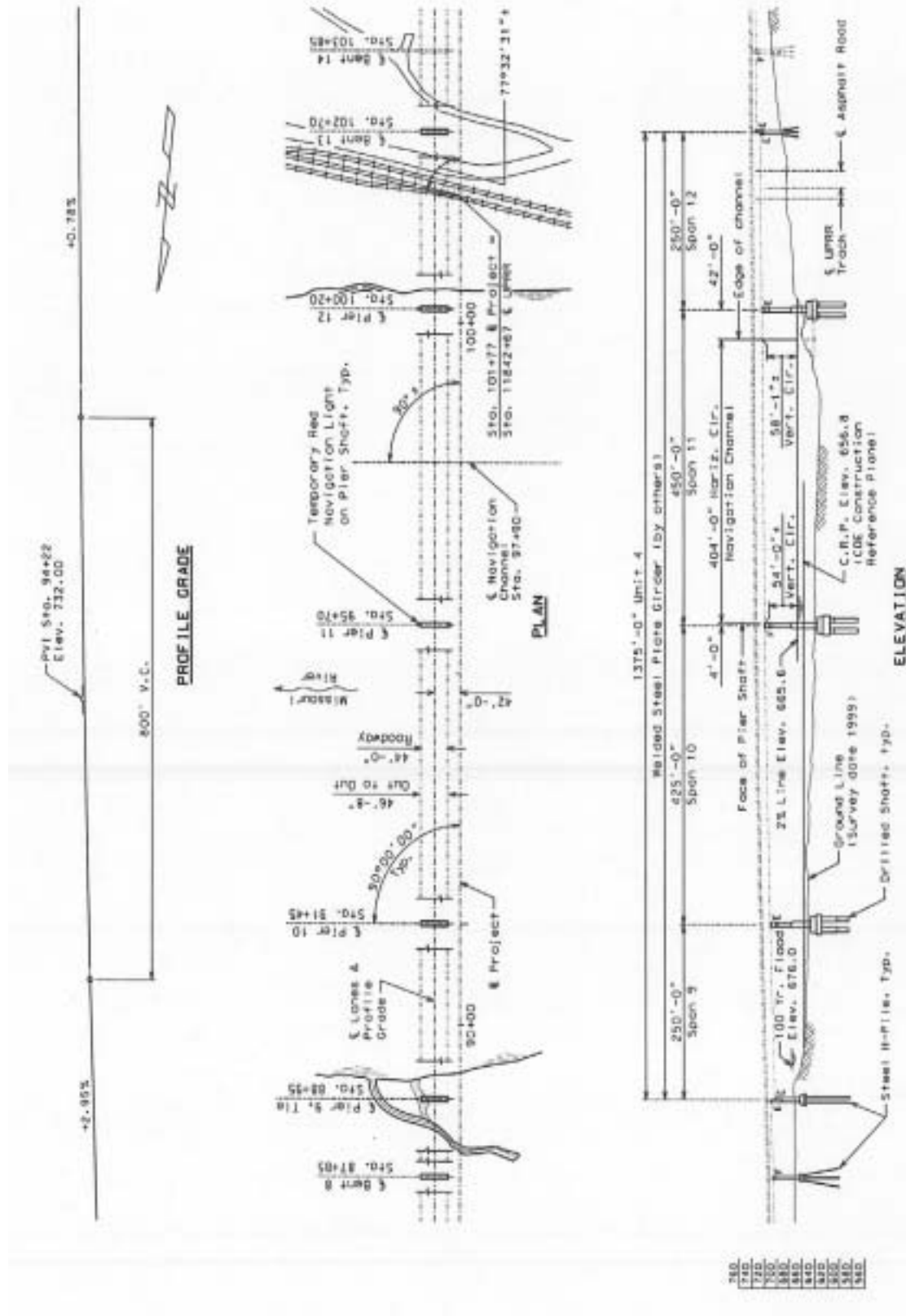
The bridge will extend from the north abutment located in the flood plain south across the flood plain and river to the south abutment located on the rolling hills on the south side of the river. The total length of the bridge is to be 2673 feet (814.7 meters) with 15 spans of various lengths. The two main river spans will be 450 and 425 feet (137.2 and 129.5 m) as shown in Figure 6.2.

Foundation elements for piers 1 through 9 and 13 though 16 were anticipated to be H-piles driven to bedrock. Drilled shafts socketed into the shale bedrock were anticipated for the three piers in the river (Piers 10, 11 and 12). An elevation view of piers 9 through 12 is shown in Figure 6.3 and the anticipated rock socket lengths and bearing values are presented in Table 6.1. Rock sockets for Piers 10, 11, and 12 will encounter the Weir Formation.

Table 6.1- Drilled shaft parameters for Piers 10, 11, and 12.

Pier	No. of Drilled Shafts	Rock Socket Length ft(m)	Rock Socket Dia. ft(m)	Design Bearing tons (MN)	Elev From-to (ft)	Allowable Unit Side Shear tsf (kPa)	Allowable Unit End Bearing tsf (kPa)
10	6	29(8.8)	6.5(2)	1000	602 - 594	0.6 (57)	0
				(8.9)	594 - 586	2.0 (191.5)	0
					586 - 570	2.6 (249)	25(2,394)
					570 - 560	1.6 (153.2)	25(2,394)
					560 - 550	2.0 (191.5)	25(2,394)
11	6	54(16.5)	6.5(2)	1524	608 - 585	0.6 (57)	0
				(13.6)	585 - 564	1.4 (134.1)	0
					564 - 555	2.0 (191.5)	25(2,394)
12	6	46(14)	6.5(2)	1,100	631 - 602	0.9 (86.2)	0
				(9.8)	602 - 585	2.0 (191.5)	0
					585 - 570	2.0 (191.5)	25(2,394)

Figure 6.3- Elevation view of bridge showing Piers 9 thru 12 and the location of the test shaft.

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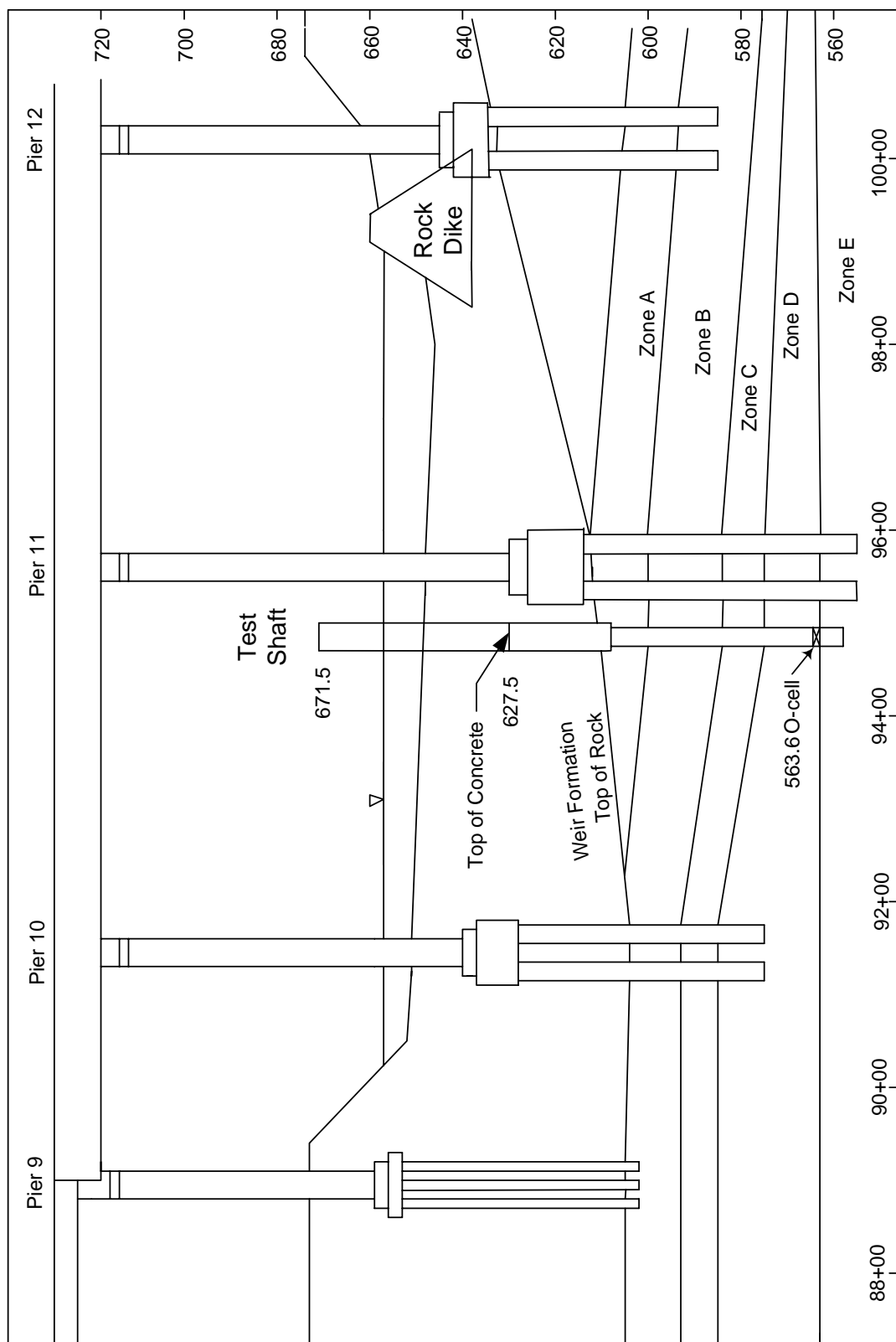
6.2 Geology of the Area

The underlying bedrock is of lower Pennsylvanian Age, Desmoinesian Series, Cherokee Group. The Cherokee Group contains most of the mineable coal beds in Missouri. The group is divided into the Krebs and Cabaniss Subgroups and the rock sockets for the drilled shafts are planned in the Cabaniss Subgroup.

The Cabaniss Subgroup consists of sandstone, siltstone, underclay, limestone, and coal beds. The subsurface investigation for the river piers encountered only the Weir Formation of the Cabaniss Subgroup. The subsurface investigation for the piers on the South river bank encountered (from the base upwards): the Weir Formation, the Tebo Formation, the Scammon Formation, and the Mineral Formation. The Weir Formation is described in the following paragraphs and the location of the rock socket for the test shaft at Pier 11 with respect to the formations is shown in Figure 6.3.

The Weir Formation is composed of (from the base upward) of shale, coarse grained sandstone, irregular bedded limestone, black carbonaceous shale, the Waverly Coal Bed, black carbonaceous shale, coal, micaceous siltshale, underclay, one to two coal beds, shale , limestone about 1 foot (0.3 m) thick, micaceous siltshale, underclay, and the Weir Pittsburg Coal bed. The top of the Weir Formation varied from elevation 625.0 to 629.4. The Weir Formation was encountered from about elevation 555 to 610 ft at Pier 11 and averages about 55 feet (16.8 m) in thickness at this location. The bedrock profile at Pier 11 was divided into 5 zones based on trends in material strength and properties.

Weir Zone A: Zone A was encountered from elevation 600 to 609 ft and consisted of gray to purple claystone and greenish-gray clay shale. Numerous slickensides were observed in the NX core. The slickensides may be attributed to natural faulting or faulting caused by the collapse of a mine. SPT blow counts in Zone A averaged 100 blows in 8 inches (20.3 cm). Liquid limits (LL) varied from 36 to 40 and the plasticity index (PI) varied from 15 to 20. Jar slake tests performed on this material produced a jar slake index of 2 as shown in Figure 6.4. Results of jar slake tests performed on various zones of the Weir Formation are summarized in Table 6.2.
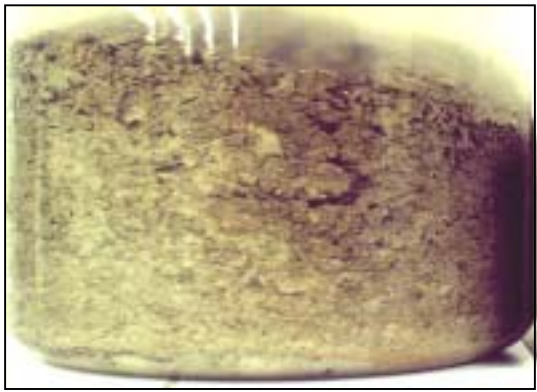
a. Slake Index (2)   b. Slake Index (2)

Figure 6.4- Range of jar slake index test results for Zone A Weir Formation: (a) elevation 606.1 ft., slake index (2), (b) elevation 602.6 ft., slake index (2).

Table 6.2-Summary of jar slake index test results for Weir Formation at Waverly site.

Formation	Zone	Elevation	1440
		(ft)	(min)
Weir	A	606.1	2
Weir	A	602.6	2
Weir	B	597.1	1
Weir	B	590.6	3
Weir	B	587.5	3
Weir	C	581.9	5
Weir	C	578.5	6
Weir	D	571.5	5
Weir	D	565.2	3

Weir Zone B: Zone B was encountered from elevation 584.7 to 600 ft and consisted of (from the base upwards): gray clay shale, gray micaceous siltshale, a gray claystone (underclay), and two separate coal layers. SPT blow counts in Zone B averaged 100 blows in 5.75 inches (14.6 cm). Atterberg limits varied from a LL of 31 with a PI of 12 in the siltshale to a liquid limit of 48 and a PI of 21 in the underclay. Jar slake indices for this material varied from a jar slake index of 1 in the underclay to a jar slake index of 3 in the siltshale as shown in Figure 6.5.

a. Slake Index (1)   b. Slake Index (3)

Figure 6.5- Range of jar slake index test results for Zone B Weir Formation: (a) elevation 597.1 ft., slake index (1), (b) elevation 590.6 ft., slake index (3).

Weir Zone C: Zone C was encountered from elevation 574 to 584.7 ft and consisted of black shale and the Waverly Coal Bed. The Waverly coal was mined in the early part of the twenty century. SPT blow counts in Zone C averaged 100 blows in 4 inches (10.2 cm). Liquid limits varied from 29 to 30 and the PI varied from 4 to 9. Jar slake indices for the black shale varied from a jar slake index of 5 to 6 as shown in Figure 6.6.

a. Slake Index (5)   b. Slake Index (6)

Figure 6.6- Range of jar slake index test results for Zone C, Weir Formation: (a)
         elevation 581.9 ft., slake index (5), (b) elevation 578.5 ft., slake index (6).

Weir Zone D: Zone D was encountered from elevation 564 to 574 ft and

consisted of black carbonaceous shale. SPT blow counts in Zone D averaged 100 blows

in 3 inches (7.6 cm). Liquid limits varied from 30 to 34 and the PI varied from 12 to 15.

Jar slake test results for the black shale varied from a jar slake index of 3 to 5 as shown in

Figure 6.7.
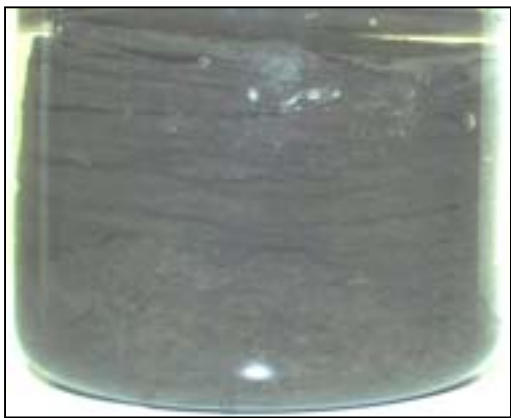
a. Slake Index (3)   b. Slake Index (5)

Figure 6.7- Range of jar slake index test results for Zone D, Weir Formation: (a)
         elevation 565.2 ft., slake index (3), (b) elevation 571.5 ft., slake index (5).

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Weir Zone E: Zone E was encountered from elevation 555 to 564 ft and

consisted of a coarse grained sandstone and fossiliferous limestone. At Pier 11, an

unconfined compressive strength of 5,429 kPa (56.7 tsf) was reported for Zone E. RQD

values for this stratum varied from 72 to 100 percent.

6.3 Unconfined Compressive Strength of Bedrock at Pier 11

Bedrock samples were taken with a standard split spoon and a NX sized double

core barrel. The core was logged with the amount of core recovered and the RQD being

noted in the limestone and sandstone layers. The core was returned to Jefferson City to

be photographed and for further testing. Unconfined compressive strengths of the rock

cores for Pier 11 varied from 1.6 to120.2 tsf (153.2 to 11,510 kPa) as shown in Table 6.3.

Since the unconfined compressive strength data was limited for Zone E, additional data

from Pier 10 was used to calculate the average q_u and the standard deviation. Zones A

and B had the lowest unconfined compressive strengths of all the shale strata while Zone

C had the highest unconfined compressive strengths for the shales. Zone D had

intermediate unconfined strengths that were approximately double that of Zones A and B.

Zone E, which is composed of sandstone and limestone had the highest overall strengths,

far exceeding the strength of the shale strata.

Table 6.3- Unconfined compressive strengths for Pier 11.

Strata	Elev.	Avg q_u		Range	Std. Dev.
Zone	ft.	MPa	tsf	tsf (MPa)	tsf (MPa)
A	600 – 609	0.43	4.5	1.6 (0.15)-7.2 (0.69)	1.5 (0.14)
B	584.7 – 600	0.47	4.9	2.1 (0.20) - 10.2 (0.98)	2.5 (0.26)
C	574 - 584.7	2.36	24.6	9.2 (0.88) – 38.1 (3.65)	15.2 (1.46)
D	564 – 574	0.89	9.3	2.1 (0.20) – 18.8 (1.80)	4.7 (0.45)
E	555 - 564	6.56	68.5	17.7 (1.69) – 120.2 (11.51)	33.4 (3.20)

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6.4 Foundation Design

Drilled shafts socketed into bedrock were chosen for the foundations of the piers in the river. Drilled shafts were chosen due to the thickness of the alluvium and the high potential depth of scour. Due to the alternating layers of shale, sandstone, siltstone, coal, and underclay the rock socket design was originally based on side resistance and ignored end bearing. The design load for the shafts at Pier 11 was estimated to be about 1,525 tons (13.6 MN). The allowable side friction, for 6.5 foot (1.98 m) diameter rock sockets was determined following procedures by Horvath and Kenney (1979). Based on these calculations the required socket lengths would be 63 feet (19.2 meters) and would be very costly. It was therefore decided to allow the design consultant to use some end bearing at Pier 11 in addition to side shear.

Due to the questionable nature of the shale bedrock, the importance of the mid-river pier (Pier 11), and the large design load, MoDOT recommended an Osterberg cell load test be performed. Since the cost and time required to complete the Osterberg cell load test during the design stage was prohibitive, it was decided to conduct the test on a production shaft. The test shaft location and Pier 11 are shown in Figure 6.3.

6.5 Construction of Test Shaft

The Osterberg cell load test was to be performed on one of the 6 "production" shafts at Pier 11 in the middle of the river as shown in Figure 6.8. A 26 inch (660 mm) Osterberg cell with a capacity of 1800 tons (16 MN) in each direction was chosen to test the production shaft to twice the design load. The Osterberg cell would be located about 5.6 feet (1.7 m) above the base of the rock socket at the interface between Zones D and E as shown in Figure 6.9.

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Figure 6.8- Pier 11 at Waverly site.

The test shaft was constructed by Jensen Construction Co. using a 9270 Series American crane with a Hain twin drill assembly as shown in Figure 6.10. Jensen mobilized and installed a 9 feet (2.74 m) diameter temporary casing almost to the top of rock using a vibratory hammer and casing clamp on September 9, 2002. The contractor began excavating the overburden material on September 11, 2002. After the overburden was removed, 84 inch (2134 mm) permanent casing was seated into the shale bedrock as shown in Figure 6.11. A 48 inch (1220 mm) diameter pilot hole was then excavated to about 10 feet (3m) above the planned tip elevation of the rock socket (elevation 555 ft). A NX size core was drilled at the bottom of the pilot hole in order to evaluate the material below the rock socket. MoDOT requires a NX size foundation test hole to be drilled after excavation of the rock socket for shafts that derive axial capacity in end bearing. Since the contractor drilled a pilot hole, the NX size foundation test hole was allowed to be

completed before the rock socket was completely excavated. The purpose of the pilot

holes was to limit the time the side walls of the rock socket were exposed to drilling

slurry.
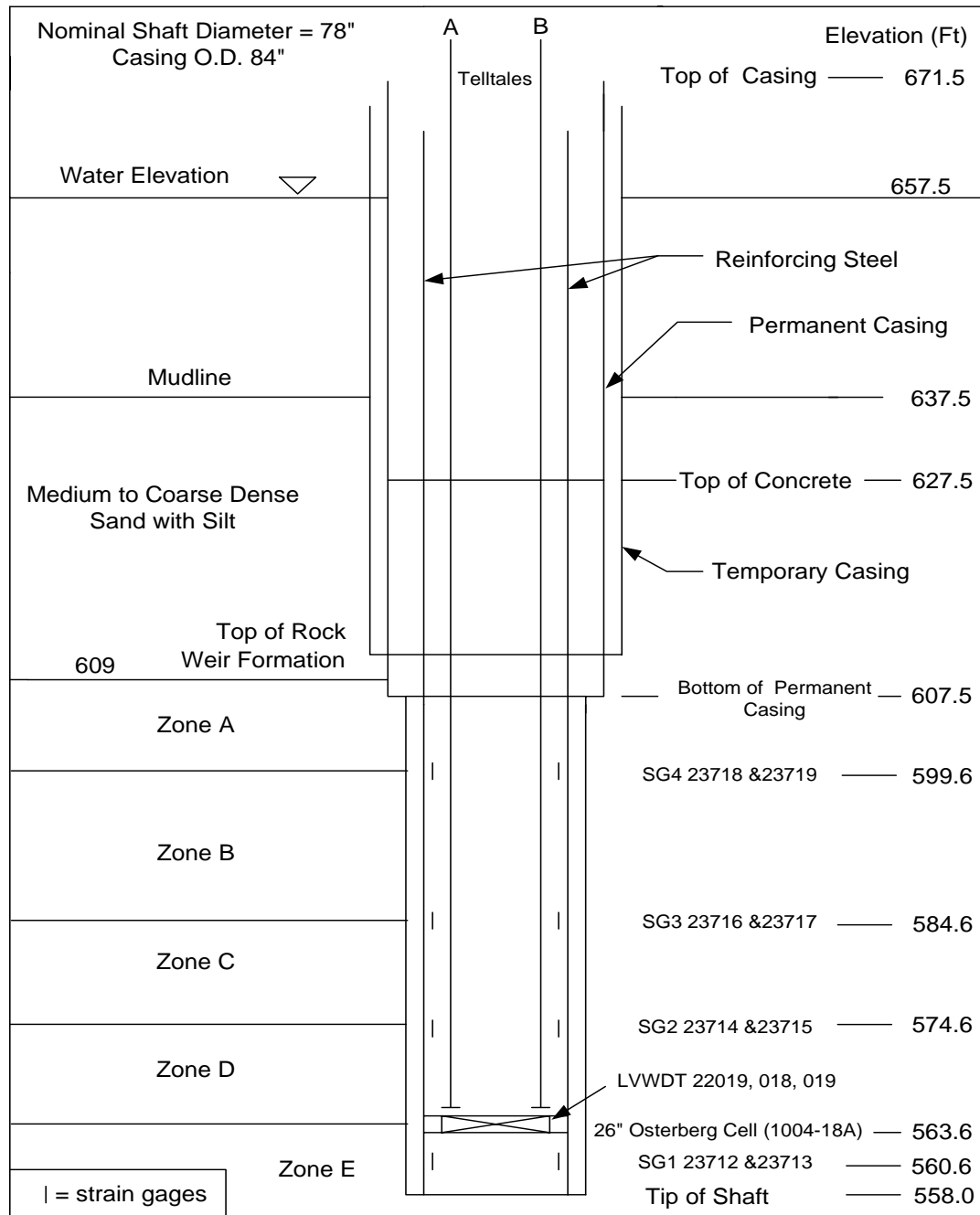
Figure 6.9- Schematic of the test shaft at Pier 11.

Figure 6.10- American 9270 Series crane with a Hain twin drill, drilling rock socket at Pier 12, existing bridge in background.

Figure 6.11- Temporary outer casing, inner permanent casing, and casing clamp at Pier 12 (Pier 11 is in the background).

Prior to drilling the rock socket to full diameter, a polymer slurry was introduced into the hole to help keep the shale from degrading. The polymer slurry, known as Super Mud, was supplied by the Polymer Drilling Systems Company. The rock socket was drilled using a bullet tooth rock auger and a 78 inch (1980 mm) core barrel as shown in Figures 6.12 and 6.13. Excavation of the rock socket began on September 19, 2002 with a 78 inch (1980 mm) bullet tooth rock auger. The excavation advanced to elevation 558 ft., where a hard layer of limestone was encountered. Although the rock socket was about 3 feet (0.9 m) above the planned tip elevation, MoDOT approved the socket. The socket was cleaned with a cleanout bucket and the cleanliness of the rock socket bottom was inspected with an underwater video inspection system shown in Figure 6.14.

Figure 6.12- Bullet tooth rock auger used to excavate rock socket at Waverly test site.

Figure 6.13- Core Barrel used to excavate rock socket at Waverly test site.

Figure 6.14- Miniature shaft inspection device (Mini-SID) used to inspect bottom of rock sockets at Waverly bridge site.

The rebar cage with the Osterberg load cell shown in Figure 6.15 was placed in the socket and an attempt was made to place the concrete in the socket on the September 20, 2002. Due to clogging of the 8 inch (203mm) tremie the concrete pour could not be completed. The rebar cage and about 20 cubic yards (15 cubic meters) of fluid concrete was removed, and the socket was cleaned. Holes in the top and bottom plate adjacent to the O-cell^{TM} were enlarged to allow a larger 12 inch (305 mm) tremie to be used with the approval of Loadtest. The shaft was then completed on the September 21, 2002 still within the required 72-hour time limit.

Figure 6.15- Rebar cage with Osterberg load cell.

The slump of the concrete was about 1 inch (25 mm) when it arrived on the job. Plastizer "Super P" was added to the concrete and the slump was about 7.5 to 8 inches (195 mm) when it was poured into the shaft. Test cylinders made of the shaft concrete were tested to a compressive strength of 7520 psi (51.8 MPa) on the day of the Osterberg cell load test. Loadtest personnel arrived on the site and completed the load test on September 30, 2002.

6.6 Load Test Setup and Procedures

The load test for the production shaft was performed by Loadtest Inc. on September 30, 2002. A schematic of the test shaft with associated instruments is shown in Figure 6.16; a summary of shaft dimensions is given in Table C.1 in Appendix C. The 26 inch (660 mm) diameter O-Cell^{TM}, with its base located 5.6 feet (1.7 m) above the tip of the rock socket, was pressurized to assess the combined end bearing and side shear below the O-Cell^{TM} and side shear above the cell. The O-Cell^{TM} was pressurized in 23 even increments of 600 psi (4,137 kPa) to a maximum O-Cell^{TM} pressure of 13,800 psi (95,147 MPa), which corresponds to a load of 2525 tons (22.5 MN) in each direction. At this load, the capacity of the Osterberg cell had been exceeded by more than 40 percent and more than twice the design load had been achieved. The O-Cell^{TM} was then unloaded in 4 equal increments and the test was concluded. The applied load increments followed procedures in ASTM D1143- Standard Test Method for Piles Under Static Axial Compressive Load.
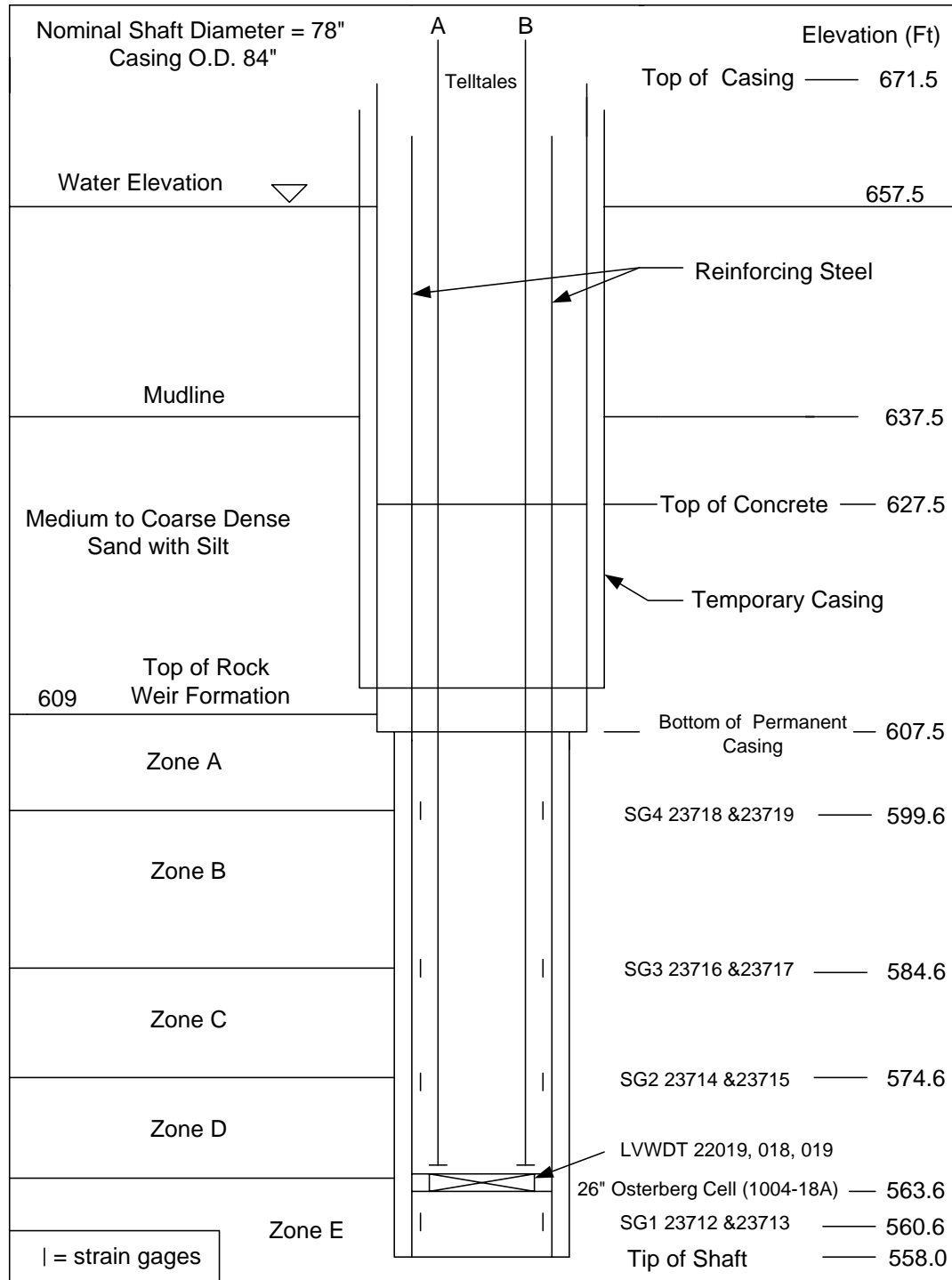
Figure 6.16- Schematic of test shaft showing location of instrumentation.

Expansion of the O-Cell^{TM} was measured by three LVWDTs (Geokon Model 4450 Series) positioned between the lower and upper plates of the O-Cell^{TM}. Telltales were inserted in pre-installed steel pipes from the upper plate of the O-Cell^{TM} to the top of shaft. Two LVWDTs (Geokon 4450 Series) attached to the telltales were used to measure compression of the shaft between the O-Cell^{TM} and the top of the shaft. Two additional LVWDTs (Geokon 4450 Series) were attached to the reference beam to measure top of shaft movement.

Strain gages were used to assess load transfer in the shaft above and below the O-Cell^{TM}. Four levels of sister bar vibrating wire strain gages, with two sister bars at each level, were installed in the shaft at the location of changes in strata as shown in Figure 6.16.

6.7 General Test Results

The results of the Osterberg cell load test at the Waverly site are presented in this section. Detailed load test data is provided in Appendix C. The measured load-displacement response for the load test on the test shaft is shown in Figure 6.17. The maximum gross load applied to the base of the shaft was equal to 2525 tons (22.5 MN) and occurred at load interval 1L-23. At this point, the O-Cell^{TM} had expanded 0.122 inches (3.10 mm) with 0.043 inches (1.10 mm) of upward movement and 0.078 inches (1.99 mm) of downward movement. The maximum net load applied to the upper portion of the shaft was equal to 2525 tons (22.5 MN) minus the buoyant weight of the shaft 115.3 tons (1.03 MN), for a net applied load of 2410 tons (21.4 MN).
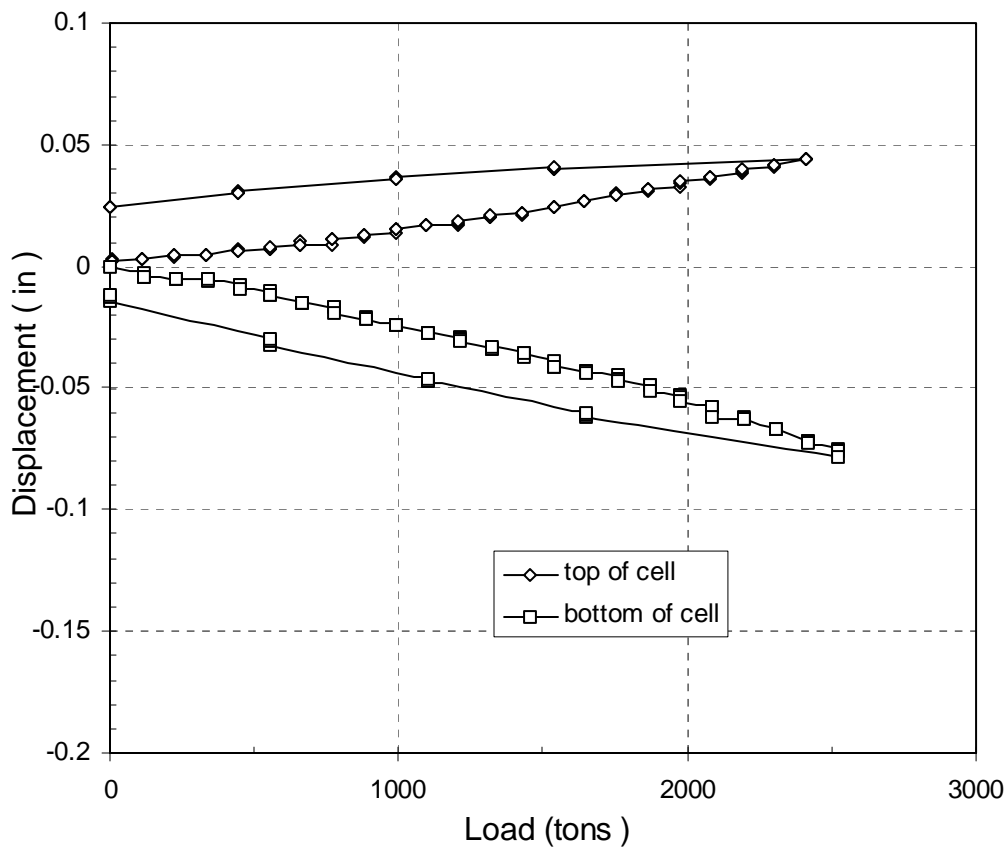
Figure 6.17- Measured load-displacement curves for test shaft at Waverly site.

Equivalent top-down load-displacement curves for the Waverly test shaft are shown in Figure 6.18. The "measured" equivalent top-down load-displacement curve indicates a settlement of approximately 0.078 inches (1.99 mm) at the maximum load of 4935 tons (43.9 MN). When adjusted for additional elastic compression that would occur in a top-down load test, a shaft loaded from the top with a load of 4,935 tons (43.9 MN) would settle about 0.26 inches (6.6 mm) of which 0.18 inches (4.6 mm) is estimated elastic compression. The equivalent top-down load-displacement curves are essentially linear over the range of loads shown, which indicates that the shaft had additional capacity beyond the load applied in the load test.
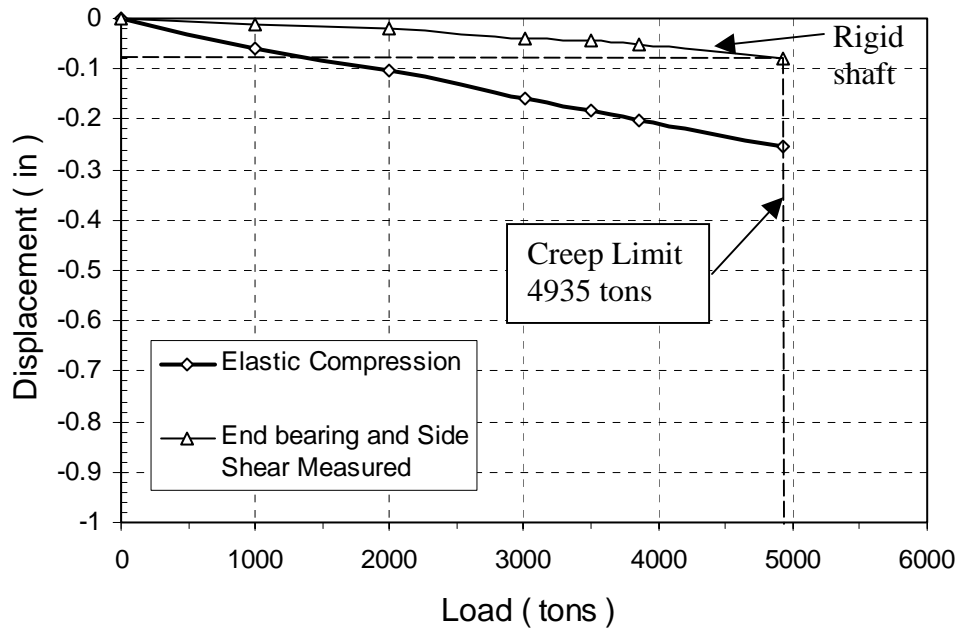
Figure 6.18- Equivalent top-down load-displacement curves for Waverly test shaft.

The distribution of axial force with elevation at various load increments was generated from strain gage data, the equivalent modulus of the shaft, and the cross-sectional area of the shaft as described in Chapter 3 (Eq. 3.7). The distribution of axial force determined for several load increments is shown in Figure 6.19. The strain gage data used to compute the distribution of axial force is provided in Appendix C. On the day of the test, the concrete compressive strength was 7520 psi (51.8 MPa). This, combined with the area of the reinforcing steel and nominal shaft diameters of 84 inches (2134 mm) above the bottom of the permanent casing and 78 inches (1981 mm) below, was used to determine an average shaft stiffness (AE) of 31,800,000 kips (141,500 MN) above the bottom of the casing and 24,900,000 kips (110,700 MN) below. The shaft stiffness along with strain gage data was used to calculate the axial force at various

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elevations. The point of zero shear in Zone A (Elevation 606.5 ft) was estimated by

projecting the unit side shear transferred in Zone B to that transferred in the shaft along

Zone A.
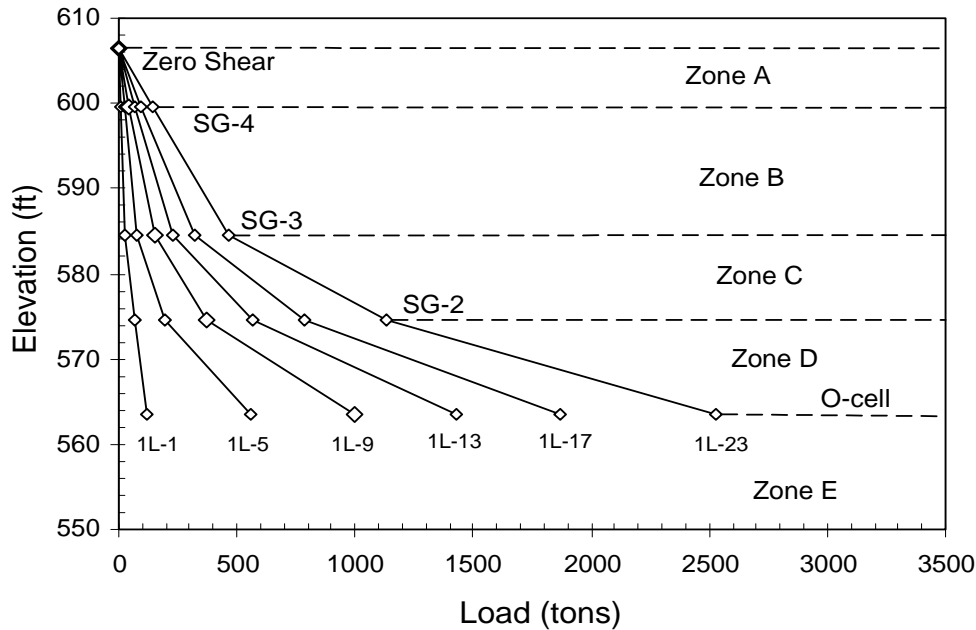
Figure 6.19- Distribution of axial force for the Waverly test shaft.

The mobilized unit side shear for each load increment is plotted versus O-cell[TM]

displacement in Figure 6.20. The unit side shear curves indicate that side shear had not

been fully mobilized in any of the shaft segments above the O-cell[TM]. The unit side shear

below the O-cell[TM] in Zone E could not be determined due to problems with the level-1

strain gages.

Values of the average mobilized unit side shear determined for various segments

of the shaft are shown in Table 6.4 and plotted in Figure 6.21. These values were

calculated for the peak load, which occurred at load interval 1L-23. The unit side shear

mobilized in Zones C and D was significantly greater than that mobilized in Zones A and

B.
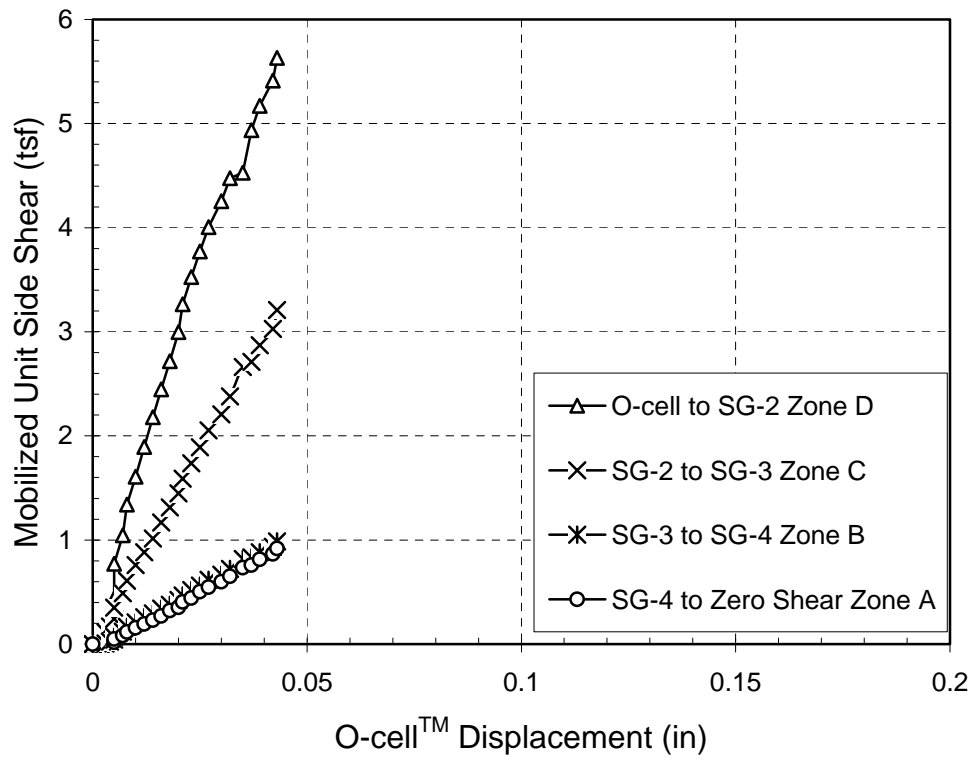
Figure 6.20- Mobilized unit side shear versus O-cell^{TM} displacement for Waverly test
shaft.
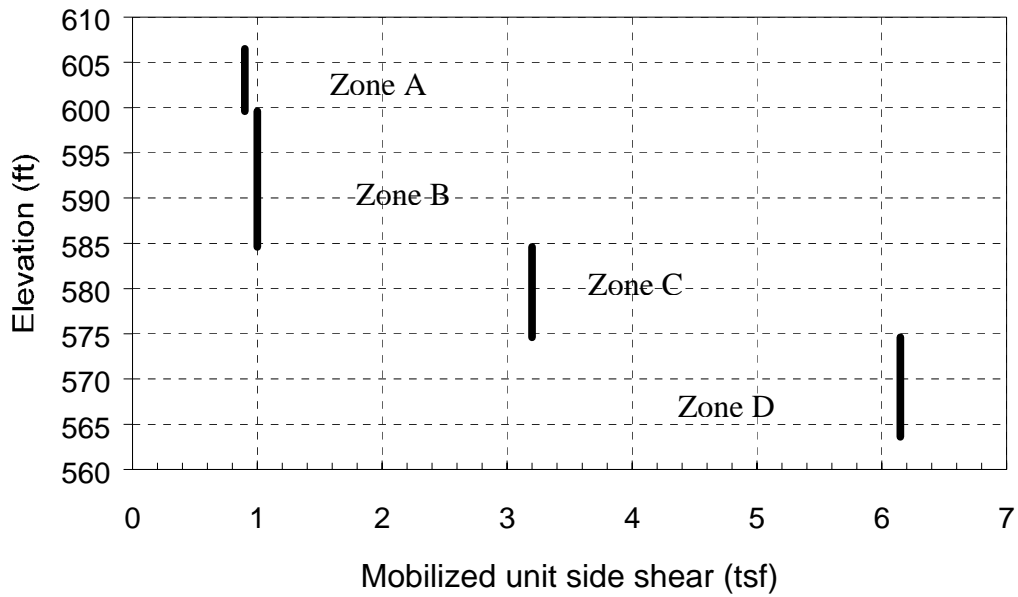
Figure 6.21- Mobilized unit side shear values calculated from strain gage data for the
Waverly test shaft.

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Table 6.4- Mobilized unit side values calculated from strain gage data for the Waverly test shaft.

Load Transfer Zone	Zone	Elevation (ft)	Unit Side Shear	
			kPa	tsf
0 shear to SG-4	A	606.5 - 599.6	88	0.9
SG-4 to SG-3	B	599.6 - 584.6	94	1.0
SG-3 to SG-2	C	584.6 - 574.6	306	3.2
SG-2 to O-Cell^{TM}	D	574.6 - 563.6	587	6.15

Level-1 strain gages did not produce reliable data.

Figures 6.22 and 6.23 show creep displacements that occurred over the time interval 2 to 4 minutes after application of the load while the load was maintained constant. As shown in the figures, no creep limit was reached for either the upper or lower portions of the shaft. These data indicate that significant creep would not occur for a top loaded shaft until a load greater than 43.9 MN (4935 tons) is exceeded by some unknown amount.
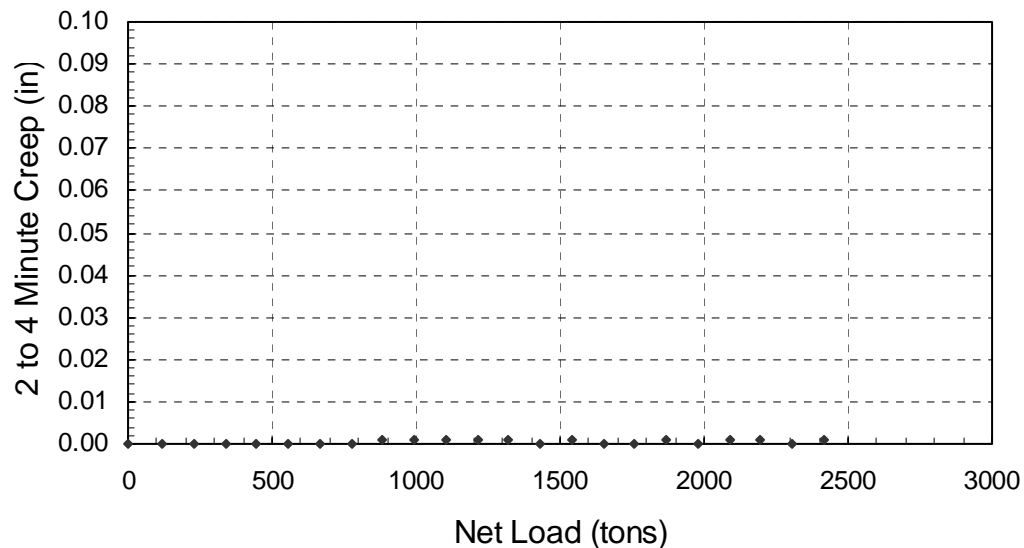
Figure 6.22- Creep displacements for the upper portion of the Waverly test shaft.
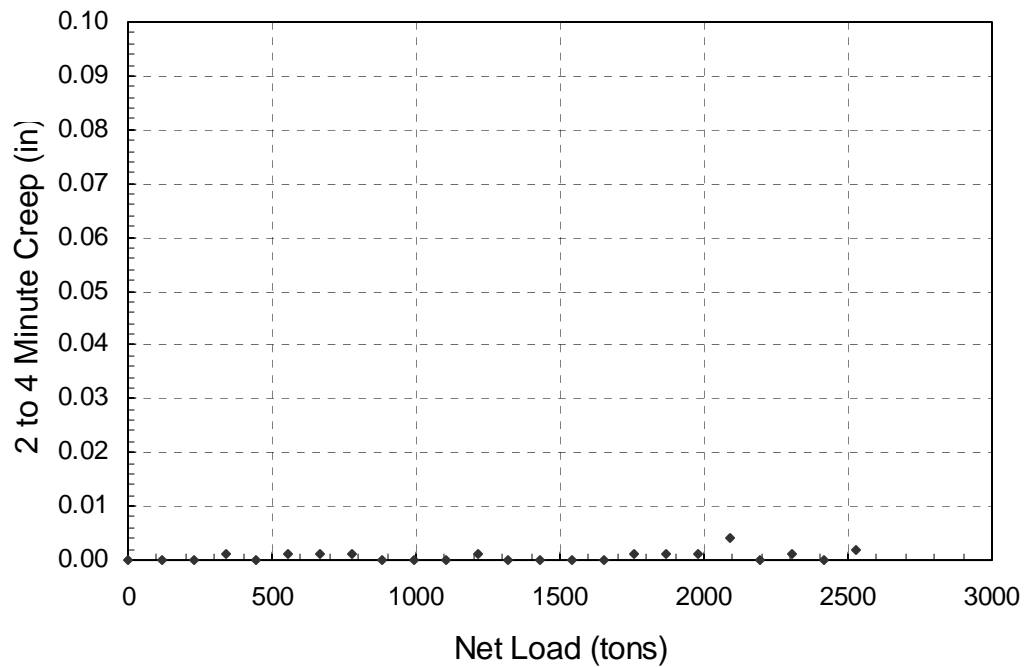
Figure 6.23- Creep displacements for the lower portion of the Waverly test shaft.

6.8 Practical Applications

Some thought was given to specifying a 34 inch (870mm) Osterberg load cell with a capacity of 3,000 tons (27 MN) and trying to achieve twice the design load (1524 tons (13.6 MN) above the Osterberg load cell. This would allow 10 feet (3.3 m) of rock socket on the remaining 5 shafts at Pier 11 to be eliminated. This would save about $80,000 in drilled shaft cost. This was offset by the additional cost of a 34 inch (870mm) cell ($40,000) and possible additional contract costs. As it was about 3 feet (0.9 m) of socket was eliminated on all 6 shafts and $29,700 was saved to offset the $70,000 cost for the original Osterberg cell load testing. The cost for the Osterberg cell load test at the Waverly site was significantly lower than the costs at either the Lexington site or the Grandview site. This is because the test was performed during the construction phase on

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a production shaft. There were therefore no costs for construction of the shaft or

mobilization. However, very little cost benefit can be realized during the construction

phase.

6.9 Summary and Conclusions

The Missouri DOT proposed to build a new bridge across the Missouri River in

the vicinity of Waverly Missouri. The foundation design for the piers in the vicinity of

the river would consists of drilled shafts socketed into bedrock. The bedrock at this

location consisted of alternating layers of clay shale, siltstone, coal, and underclay with

scattered layers of limestone and sandstone. Since the shales could not support large

axial loads in end bearing, it was decided to design the rock sockets based on side shear

only. Current design methods used by MoDOT would require exceedingly long rock

sockets that would be very expensive. Due to the questionable nature of the bedrock at

Pier 11 and the desire to use some end bearing it was decided to test a rock socketed

drilled shaft using an Osterberg cell load test. The Osterberg cell load test was performed

on a production shaft for Pier 11 in September 2002. The shaft was constructed

following lessons learned at the Lexington test site: the rock socket was excavated and

concrete placed within 72 hours and polymer slurry was used to reduce the degradation or

softening of the rock socket walls.

A general geologic description of the Waverly test site was presented in this

chapter followed by a summary of the engineering characteristics of the most pertinent

strata. The construction and testing procedures for the shaft was described, followed by

presentation of the results from the load test.

The Osterberg cell load test was successful in testing the shaft to twice the design load and assuring the foundation engineers that the main pier in the river would be safe. However, the test did not fully indicate the capacity of the rock socket in either side shear or end bearing.

CHAPTER SEVEN
EVALUATION OF DESIGN METHODS

7.1 Introduction

The current procedures used by the Missouri Department of Transportation (MoDOT) to estimate the ultimate unit side shear in shales roughly predict the ultimate unit side shear to be equal to 0.15 times the average unconfined compressive strength (q_u) of the shale. In order to achieve more economical designs and to take some of the uncertainty out of the prediction of the ultimate unit side shear, MoDOT has conducted four Osterberg cell load tests at three bridge sites. Details of the load test(s) at each site were described in Chapters 4, 5, and 6. In this chapter, the results of analyses performed to evaluate the suitability of several methods for predicting the ultimate unit side shear based on these tests are presented.

Because the load tests were originally performed and analyzed using different units, all results are presented in English units in this chapter. Rather than simultaneously presenting results in dual units, the tables and figures presented in this chapter are provided in Appendix D using SI units.

7.2 Summary of Test Results

A summary of the average unconfined compressive strengths for all strata at the three sites is presented in Table 7.1 along with the maximum measured unit side shear values determined for each strata from the load tests. Measured unit side shear values for all three sites ranged from 0.9 to 17.3 tsf (88 to 1653 kPa) for the shale strata. Measured unit side shear values reported for several of these strata do not represent ultimate values since side shear was not fully mobilized in some portions of the shafts. These values

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therefore represent lower-bounds of the ultimate unit side shear that can be achieved in

these strata as indicated in the table. In strata where the ultimate side shear was fully

mobilized, unit side shear values ranged from 3.1 to 10.7 tsf (295 to 1020 kPa).

Table 7.1- Summary of measured unit side shear values and average unconfined
compressive strength (q_u) values of shale at test sites.

Formation	Elevation of Strata	Shaft Section used to calc unit side shear	Elevation of Shaft Segment	q_u Avg.	Std. Dev.	Measured Unit side shear, $f_s^{(1)}$
	(ft)		(ft)	(tsf)	(tsf)	(tsf)
		Lexington				
Bevier (C1)	591.9–613.5	TS-2 upper cell to TOS	591.1 - 605	39.8	23.1	10.7
Bevier (C1)	591.9–613.5	TS-2 SG-3 to SG-4	596.1–599.3	39.8	23.1	>7.3$^{(2)}$
Bevier (C1)	591.9–613.5	TS-2 upper cell to SG-3	591.1–596.1	39.8	23.1	>17.3
Bevier (C2)	578.8–591.9	TS-2, stage 1, lower cell to SG-2	578 – 586.2	31.3	26.8	9.2
Verdigris (D)	569.3–578.8	TS-1A, SG-4 to TOS	572.5–577.2	12.7	13.0	>4.1
Verdigris (D)	569.3–578.8	TS-1A, SG-3 to SG-4	567.6–572.5	12.7	13.0	>10.1
Croweburg (E)	554.5–569.3	TS-1A, O-cell to SG-2	559.4–564.4	17.9	16.2	7.6
Croweburg (E)	554.5–569.3	TS-1A, SG-1 to O-cell	556.5-559.4	17.9	16.2	10.1
		Grandview				
W. Chanute	927.2- 934	SG-6 to TOS	927 – 934	9.8	2.3	>3.1
Chanute	916.5–927.2	SG-5 to SG-6	921 - 927	7.2	3.5	3.1
Chanute	916.5–927.2	SG-4 to SG-5	916.5 – 921	7.2	3.5	4.8
Cement City	911.3– 916.5	SG-3 to SG-4	911.5–916.5	386.6	179.8	>17.2
Quivira	904.8– 911.3	O-cell to SG-3	905 – 911.5	14.9	3.0	4.8$^{(3)}$
Westerville	898.1– 904.8	SG-1 to O-cell	898 - 905	657.1	311.9	>24.0
Wea	< 898.1	Tip to SG-1	893.4 – 898	24.1	8.7	5.9
		Waverly				
Weir (A)	600 – 609	SG-4 to O shear	599.6–606.5	4.5	1.5	>0.9
Weir (B)	584.7 – 600	SG-3 to SG-4	584.6–599.6	4.9	2.5	>1.0
Weir (C)	574 – 584.7	SG-2 to SG-3	574.6–584.6	24.6	15.2	>3.2
Weir (D)	564 – 574	O-cell to SG-2	563.6–574.6	9.3	4.7	>6.15
Weir (E)	555 - 564			68.5	33.4	

(1) Values reported are ultimate values unless otherwise indicated.
(2) The symbol ">" indicates that the ultimate unit side shear was not reached during test, value reported is maximum value during test.
(3) Assumed ultimate unit side shear.

Figure 7.1 shows the measured unit side shear values plotted as a function of the average unconfined compressive strength for the respective strata. In the figure, closed symbols are used to represent data where the ultimate unit side shear was fully mobilized while open symbols are used to represent data where the ultimate unit side shear was not fully mobilized. Also shown are lines representing unit side shear values equal to 0.15 q_u (roughly equivalent to MoDOT's current design procedure) and 0.30 q_u. As shown in the figure, the line for $f_S = 0.15\ q_u$ is well below the ultimate unit side shear values determined from all of the load tests. The line representing $f_S = 0.30$ is a better fit but tends to under predict f_S for values of q_u less than 20 tsf and slightly over predict f_S for values of q_u greater than 20 tsf.
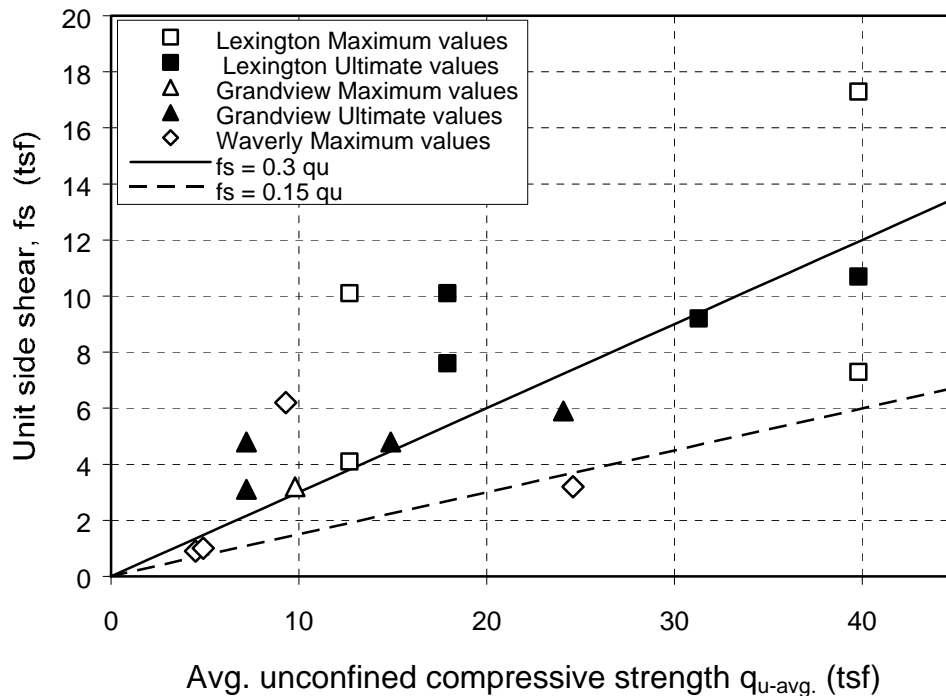
Figure 7.1- Unit side shear versus average q_u.

7.3 Interpreted Alpha Factors

The simplest method for interpreting and designing for side shear in drilled shafts is to represent the capacity as

$$f_S = \alpha \cdot q_u \qquad\qquad (7.1)$$

where α is an empirical proportionality factor to account for load transfer in the shaft. It is important to note that α, as used in Equation 7.1, is defined with respect to the unconfined compressive strength, q_u. This is a source of some confusion given that α is also frequently used as the proportionality factor relating unit side shear to the undrained shear strength ($q_u / 2$) in clay soils. The two values are not the same. However, previous investigators (e.g. O'Neill et al. 1996) have used α for weak rock ("intermediate geomaterials") in a similar manner so this convention has also been used here. Rearranging Equation 7.1, back-calculated α values can be computed by dividing the measured values of unit side shear by the unconfined compressive strength of the stratum. Alpha (α) values computed in this manner using average values of q_u for each stratum are summarized in Table 7.2 and plotted versus average values of q_u in Figure 7.2. In the figure, back-calculated α values for the shale ranged from 0.13 to 0.80 for all the sites. For sites where the ultimate unit side shear was mobilized in the shale, α ranged from 0.24 to 0.67. Osterberg (1992) has reported that previous O-cell^{TM} tests in weak rock have produced values of unit side shear as high as 0.3 to 0.5 times q_u. Data from Williams et al. (1980) show values of α as high as 1.0 or larger (Figure 2.2). Alpha (α)

values greater than 0.5 indicate that the unit side shear is greater than the undrained shear strength of the shale (q_u/2). The values reported for α greater than 0.5 are believed to be primarily due to variability in the unconfined compressive strength as considered in more detail below. The high α values may also be attributed to the roughness of the sockets resulting from the use of a rock auger to excavate shafts in shale. In rough sockets, the interface between the shaft and concrete may tend to dilate when loaded, thereby increasing the available shear resistance above the undrained shear strength in soil or rock with ϕ > 0.
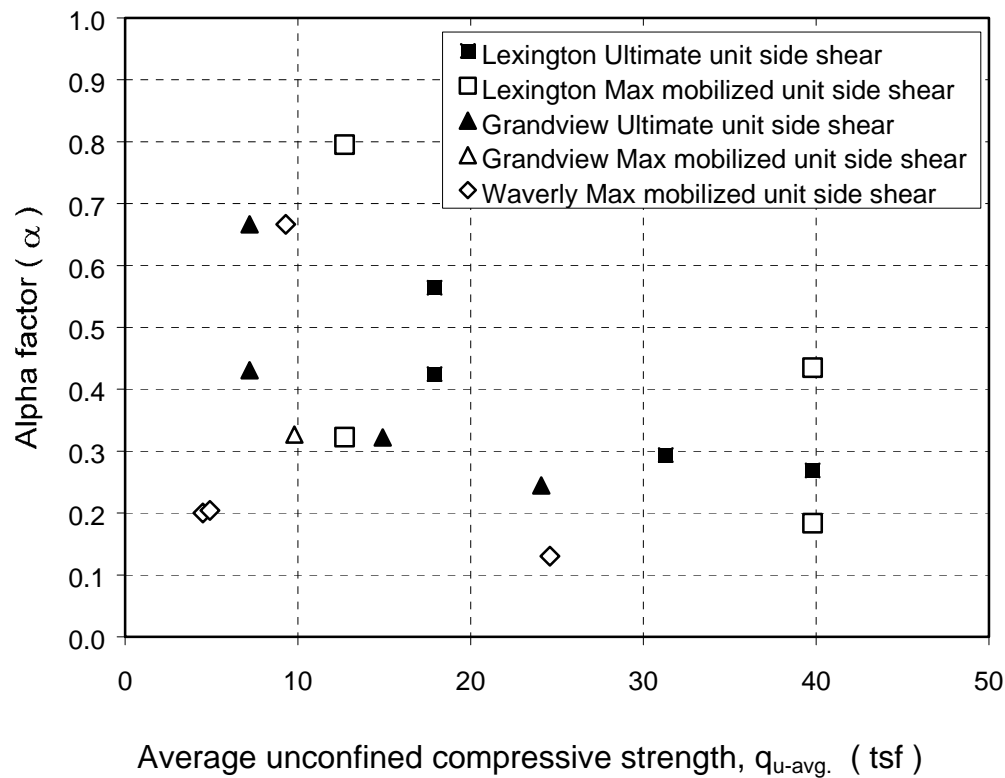
Figure 7.2- Back-calculated alpha factor (α) versus average q_u for test sites in shale.

Table 7.2- Summary of back-calculated alpha values for shale at Missouri test sites.

Formation	Shaft Section used to calc unit side shear	Elevation of shaft Segment	qu, Avg.	qu, Std. Dev.	Meas unit side shear, $f_s^{(1)}$	α Avg.	α + 1 Std. Dev.	α - 1 Std. Dev.
		(ft)	(tsf)	(tsf)	(tsf)			
	Lexington							
Bevier (C1)	TS-2 upper cell to TOS	591.1-605	39.8	23.1	10.7	0.27	0.17	0.64
Bevier (C1)	TS-2 SG-3 to SG-4	596.1–599.3	39.8	23.1	>7.3$^{(2)}$	0.18	0.12	0.44
Bevier (C1)	TS-2 upper cell to SG-3	591.1–596.1	39.8	23.1	>17.3	0.43	0.28	1.04
Bevier (C2)	TS-2, stage 1, lower cell to SG-2	578 – 586.2	31.3	26.8	9.2	0.29	0.16	2.04
Verdigris (D)	TS-1A, SG-4 to TOS	572.5– 77.2	12.7	13.0	>4.1	0.32	0.16	-13.7
Verdigris (D)	TS-1A, SG-3 to SG-4	567.6–572.5	12.7	13.0	>10.1	0.80	0.39	-33.7
Croweburg (E)	TS-1A, O-cell to SG-2	559.4–564.4	17.9	16.2	7.6	0.42	0.22	4.47
Croweburg (E)	TS-1A, SG-1 to O-cell	556.5–559.4	17.9	16.2	10.1	0.56	0.30	5.94
	Grandview							
W. Chanute	SG-6 to TOS	927 – 934	9.8	2.3	>3.2	0.33	0.26	0.43
Chanute	SG-5 to SG-6	921 - 927	7.2	3.5	3.1	0.43	0.29	0.84
Chanute	SG-4 to SG-5	916.5 – 921	7.2	3.5	4.8	0.67	0.45	1.30
Cement City	SG-3 to SG-4	911.5–916.5	386.6	179.8	>17.2			
Quivira	O-cell to SG-3	905 – 911.5	14.9	3.0	4.8$^{(3)}$	0.32	0.27	0.40
Westerville	SG-1 to O-cell	898 – 905	657.1	311.9	>24.0			
Wea	Tip to SG-1	893.4 – 898	24.1	8.7	5.9	0.24	0.18	0.38
	Waverly							
Weir (A)	SG-4 to O shear	599.6–606.5	4.5	1.5	>0.9	0.20	0.15	0.30
Weir (B)	SG-3 to SG-4	584.6–599.6	4.9	2.5	>1.0	0.20	0.14	0.42
Weir (C)	SG-2 to SG-3	574.6–584.6	24.6	15.2	>3.2	0.13	0.08	0.34
Weir (D)	O-cell to SG-2	563.6–574.6	9.3	4.7	>6.15	0.67	0.44	1.35
Weir (E)			68.5	33.4				

(1) Values reported are ultimate values unless otherwise indicated.
(2) The symbol ">" indicates that the ultimate unit side shear was not reached during test, value reported is maximum value during test.
(3) Assumed ultimate unit side shear.

Because there is variability in the value of q_u for each stratum, α-values were also

calculated for q_u equal to the mean value of q_u plus one standard deviation and the mean

value minus one standard deviation. Alpha values, calculated for the higher and lower

q_u-values are plotted in Figure 7.3. Back calculated α values from the higher q_u-values

ranged from 0.08 to 0.45 for all the sites. For sites where the ultimate unit side shear was

mobilized, back calculated α ranged from 0.16 to 0.45. Back calculated α- values for the

lower q_u-values ranged from 0.30 to as high as 5.94 for the Croweburg Formation at the

Lexington site. Negative values were also noted for the Verdigris Formation. The high

α values in the Croweburg Formation and the negative values in the Verdigris Formation

may be attributed to very large standard deviations for the unconfined compressive

strength of these strata, which are caused by the large ranges in unconfined compressive

strengths. For the Croweburg Formation, the unconfined compressive strength ranged

from 2.6 to 58.4 tsf (253 to 5,590 kPa) and for the Verdigris Formation q_u ranged from

2.3 to 46.8 tsf (218 to 4,482 kPa).

7.4 Evaluation of Design Methods

As shown in Figure 7.1, the relationship between the unit side shear and the

unconfined compressive strength is nearer to a power function than a linear relationship.

Measured values of unit side shear were compared to predicted unit side shear values

using several common design methods. The predicted unit side shear values were

calculated using the following methods:

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· Horvath and Kenney, 1979 (Eq. 2.5)

· Rowe and Armitage, 1987 (Eq. 2.9)

· Reese and O'Neill, 1988 (Eq. 2.5, 2.11, and 2.12)

· Kulhawy and Phoon, 1993 (Eq. 2.13)

· O'Neill and Reese, 1999 (Eq. 2.17 and 2.18)

where $\alpha = 0.25$ was used in the prediction of unit side shear for the method developed

by Horvath and Kenney and $\alpha = 0.45$ was used with Rowe and the Armitage method.

The unit side shear for the method proposed by Reese and O'Neill (1988) was calculated

using $\alpha = 0.21$. A mean value of $\psi = 2$ was used with the Kulhawy and Phoon method.

For each method, the ultimate unit side shear was calculated using SI units and converted

to English units. Comparisons of predicted versus measured unit side shear values are

presented in Table 7.3 and Figure 7.4 for each of these methods.

As shown in Figure 7.4, Rowe and Armitage (1987) and Kulhawy and Phoon

(1993) produced almost identical results and best fit the measured data where the unit

side shear was fully mobilized. Horvath and Kenney (1979) and Reese and O'Neill

(1988) produced similar predictions, but both methods significantly underestimated the

measured unit side shear observed in the load tests. O'Neill and Reese (1999) tended to

be even more conservative than either Horvath and Kenney or Reese and O'Neill (1988)

when the simplified method for smooth sockets was used. However, the method

proposed by O'Neill and Reese (1999) is not intended for use when the slump of the

concrete is less than 7 inches (175 mm) as was the case for the Lexington test shaft.

Predicted values obtained for the Lexington test shafts where the slump of the concrete

was about 4 inches (102 mm) were extrapolated using Equation 7.2 and Figure 7.5:

$$\sigma_n = M \, \gamma_c \, Z_i \hspace{5cm} (7.2)$$

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where M is a factor to account for the slump of the concrete, γ_c is the unit weight of

concrete in kN/m^3, and Z_i is the depth to middle of layer in meters with a maximum

depth of 40 feet (12 m).
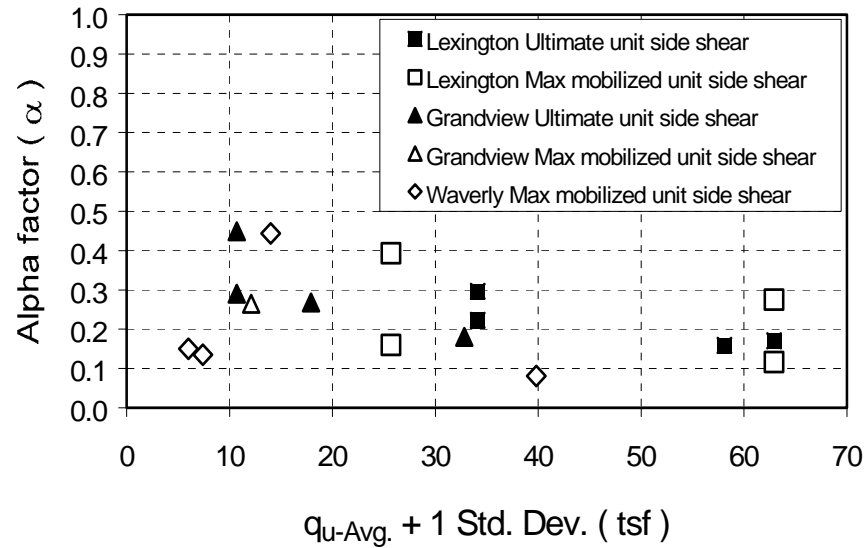
a. $q_{u\text{-avg.}}$ plus one standard deviation.
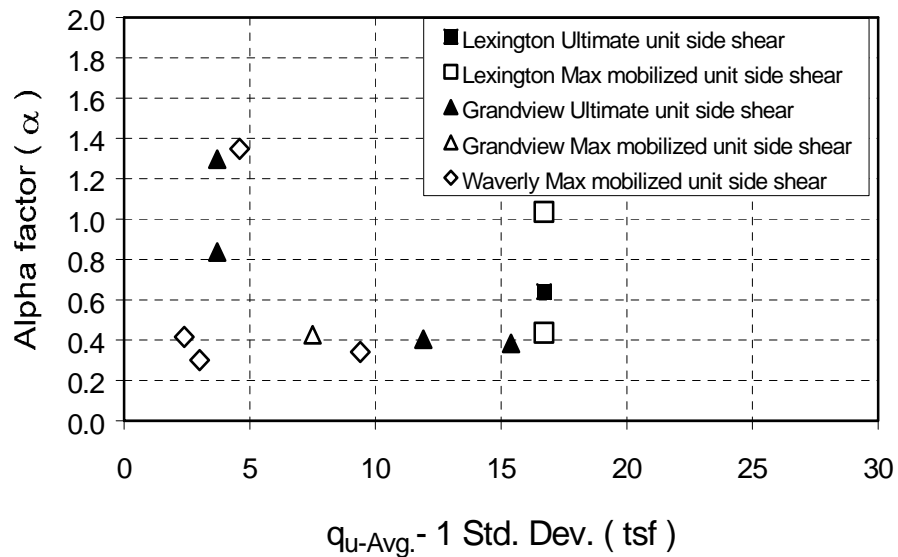
b. $q_{u\text{-avg.}}$ minus one standard deviation.

Figure 7.3- Back-calculated alpha (α) versus q_u: (a) $q_{u\text{-avg.}}$ plus one standard deviation, (b) $q_{u\text{-avg.}}$ minus one standard deviation.

Table 7.3- Comparison of measured and predicted unit side shear determined by various methods.

Formation	Shaft Segment	Elevation of Shaft Segment	q_u Avg.	Meas. unit side shear, f_s [1]	Predicted ultimate unit side shear				
					Horvath & Kenney (1979) $\alpha = 0.25$	Rowe & Armitage (1987) $\alpha = 0.45$	Reese & O'Neill (1988) $\alpha = 0.21$	Kulhawy & Phoon (1993) $\psi = 2$	O'Neill & Reese (1999) [4]
		(ft)	(tsf)	(tsf)	(tsf)	(tsf)	(tsf)	(tsf)	(tsf)
Lexington									
Bevier C1	TS-2, upper cell to TOS	591.1 - 605	39.8	10.7	5.1	9.2	4.3	9.2	2.0
Bevier C1	TS-2, SG-3 to SG-4	596.1 - 599.3	39.8	>7.3[2]	5.1	9.2	4.3	9.2	2.4
Bevier C1	TS-2, upper cell toSG-3	591.1 - 596.1	39.8	>17.3	5.1	9.2	4.3	9.2	2.4
Bevier C2	TS-2, stage 1, lower cell to SG-2	578 - 586.2	31.3	9.2	4.5	8.1	3.8	8.1	2.8
Verdigris (D)	TS-1A SG-4 to TOS	572.5 - 577.1	12.7	>4.1	2.9	5.2	3.5	5.2	1.8
Verdigris (D)	TS-1A SG-3 to SG-4	567.6 - 572.5	12.7	>10.1	2.9	5.2	3.5	5.2	1.8
Croweburg (E)	TS-1A O-cell to SG-2	559.4 - 564.4	17.9	7.6	3.4	6.2	4.9	6.1	2.5
Croweburg (E)	TS-1A SG-1 to O-cell	556.5 - 559.4	17.9	10.1	3.4	6.2	4.9	6.1	2.5
Grandview									
W. Chanute	SG-6 to TOS	927 - 934	9.8	>3.2	2.5	4.6	2.7	4.6	0.6
Chanute	SG-5 to SG-6	921 - 927	7.2	3.1	2.5	3.9	2.0	3.9	0.6
Chanute	SG-4 to SG-5	916.5 - 921	7.2	4.8	2.2	3.9	2.0	3.9	0.7
Cement City	SG-3 to SG-4	911.5 - 916.5	386.6						
Quivira	O-cell to SG-3	905 - 911.5	14.9	4.8[3]	3.1	5.6	4.0	5.6	2.0
Westerville	SG-1 to O-cell	898 - 905	657.1						
Wea	Tip to SG-1	893.4 - 898	24.1	5.9	4.0	7.1	3.3	7.1	3.4
Waverly									
Weir (A)	SG-4 to O shear	599.6 - 606.5	4.5	>0.9	1.8	3.2	1.3	3.2	1.2
Weir (B)	SG-3 to SG-4	584.6 - 599.6	4.9	>1.0	1.8	3.2	1.3	3.2	1.5
Weir (C)	SG-2 to SG3	574.6 - 584.6	24.6	>3.2	4.0	7.2	3.4	7.2	3.4
Weir (D)	O-cell to SG-2	563.6 - 574.6	9.3	>6.2	2.5	4.4	1.4	4.4	2.0
Weir (E)			68.5						

(1) Values reported are ultimate (2) The symbol ">" indicates that the ultimate unit side shear was not (3) Assumed unit (4) Method not intended for values unless otherwise indicated. reached during test, value reported is maximum value during test. side shear slump less than 175 mm (7 in)
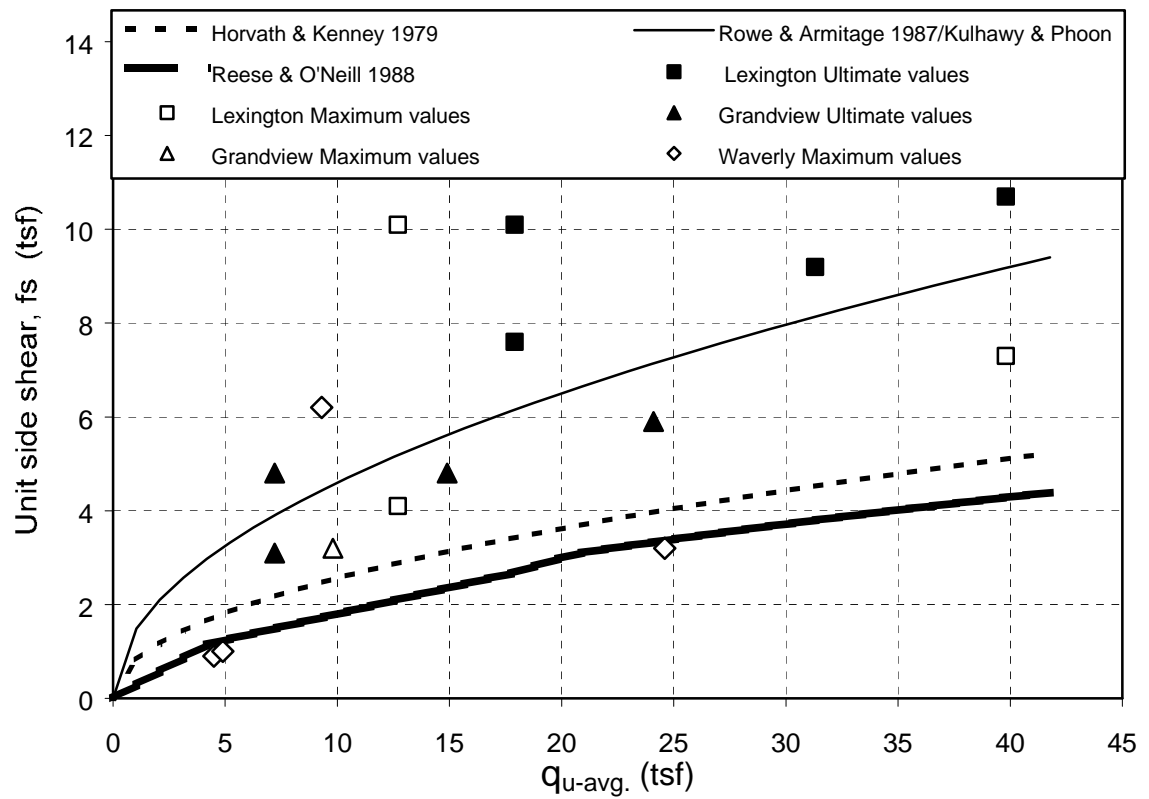
Figure 7.4- Comparison of measured unit side shear data to predicted unit side shear by several methods.
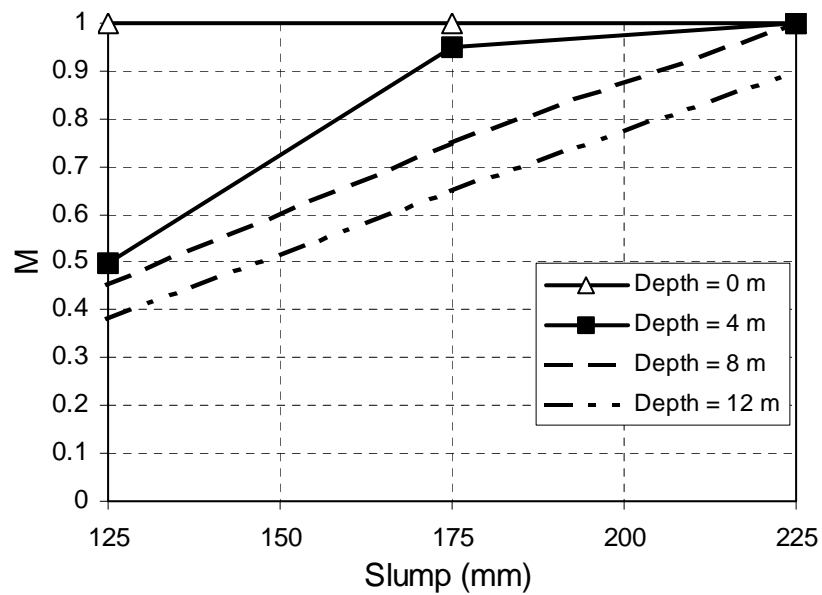
Figure 7.5- Factor M versus concrete slump (after O'Neill et al. 1996).

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Because the Rowe and Armitage method slightly over predicts values of f_S determined from load tests, a slight modification to the method is proposed for use in predicting f_S for large drilled shafts in Missouri Pennsylvanian Age shales. As shown in Figure 7.6, this adjustment produces slightly more conservative values and tends to better fit the lower bound of the measured ultimate unit side shear values determined at the three Missouri test sites. The modified Rowe and Armitage method is recommended for design of large drilled shafts in Missouri Pennsylvanian Age shales drilled with a rock auger.
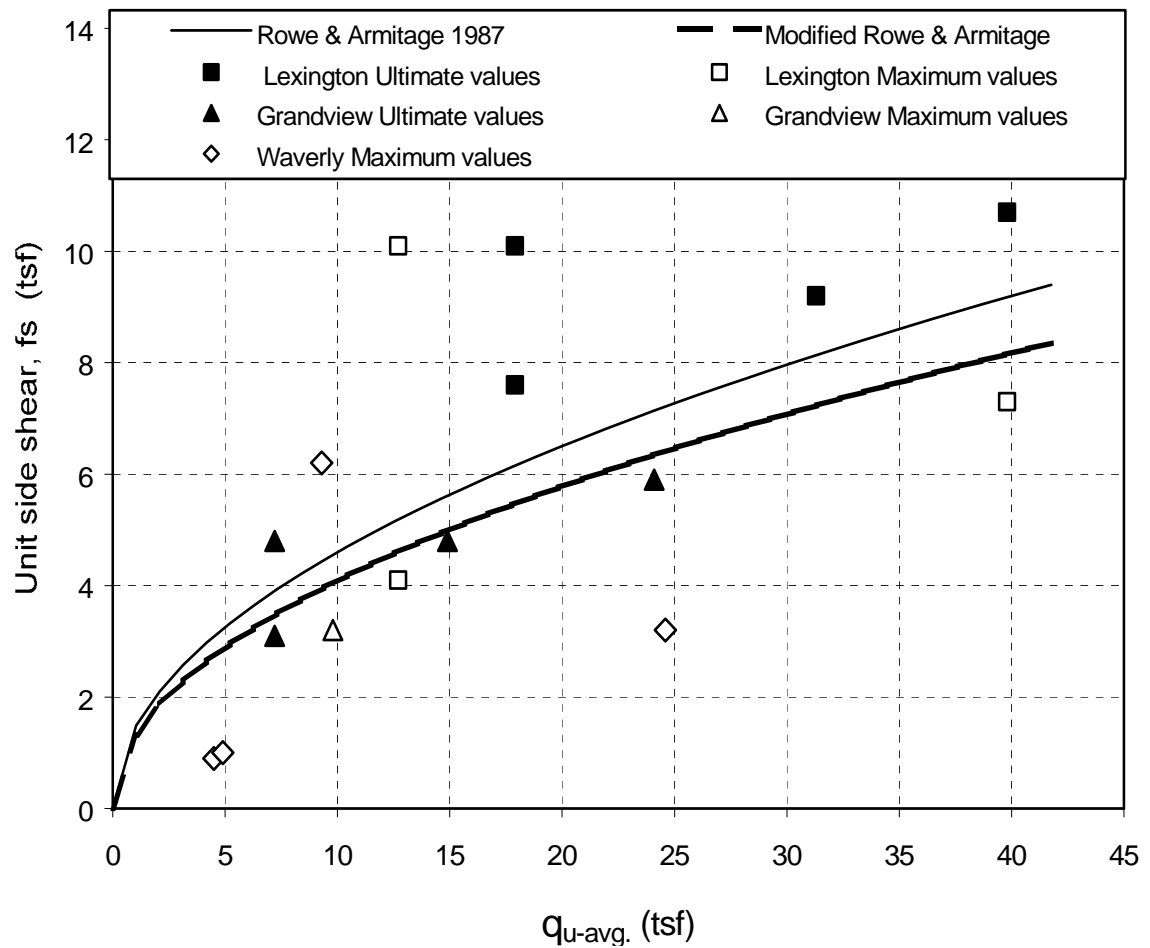
Figure 7.6-Modified Rowe and Armitage method.

Direct comparisons of predicted and measured unit side shear values are presented in Figures 7.7, 7.8, and 7.9 for the Horvath and Kenney, Rowe and Armitage, and the modified Rowe and Armitage methods, respectively. Also shown in the figures is a line that represents a factor of safety of one (a perfect prediction), and lines that represent factors of safety of 0.5 and 2.0, respectively. Points above the FS = 1 line indicate conservative predictions while points falling below this line represent unconservative predictions. As can be seen in Figure 7.7, the method proposed by Horvath and Kenney (1979) is generally conservative by a factor of approximately 2. Figure 7.8 indicates that the method proposed by Rowe and Armitage is slightly unconservative for shales with relatively low shear strength however; most of the data fell well above the line representing a factor of safety of 0.5, which suggests that the method would be acceptable if a factor of safety greater than 2.0 is used. The modified Rowe and Armitage method proposed here is slightly more conservative as shown in Figure 7.9, particularly for shales with low unconfined compressive strengths.

7.5 Summary

In this chapter, a summary and discussion of the results of four full-scale Osterberg cell load tests performed at three sites for the Missouri Department of Transportation were presented. Predicted values of f_s from several different methods were compared to the measured ultimate unit side shear values determined from the load tests. These analyses indicate that the method proposed by Rowe and Armitage (1987) most closely predicts the measured values for unit side shear. However, the method slightly over predicts f_s for relatively weak shales. A modification to the Rowe and

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Armitage method was therefore proposed to predict slightly more conservative values of

f_S for weak shales.
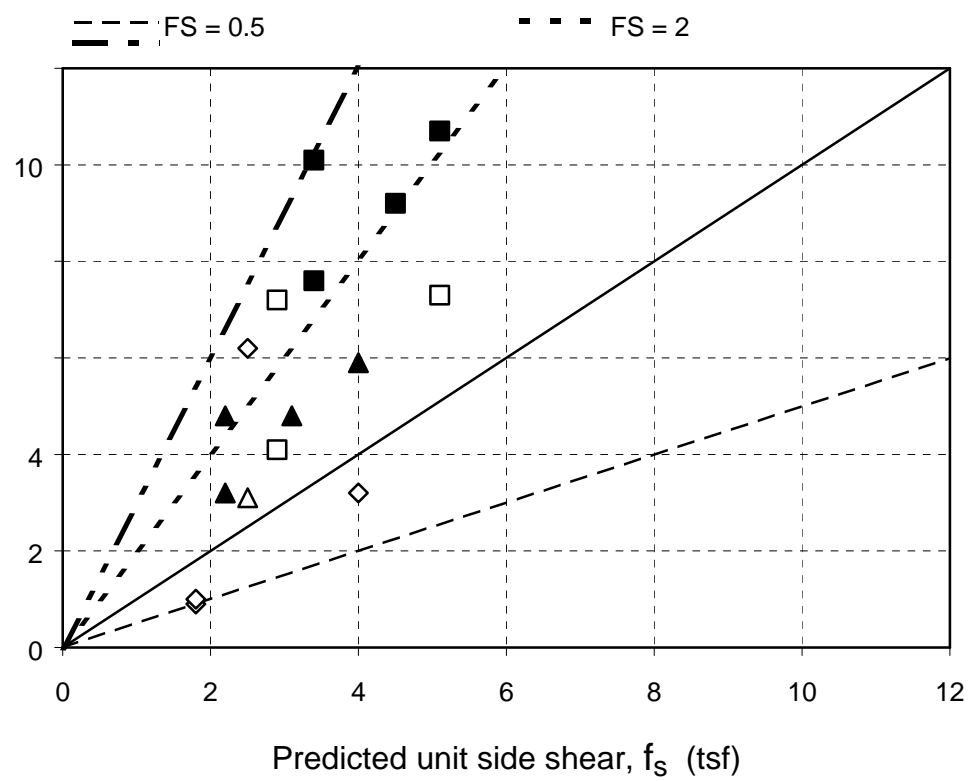
Figure 7.7- Comparison of measured and predicted unit side shear value using the Horvath and Kenney (1979) method.
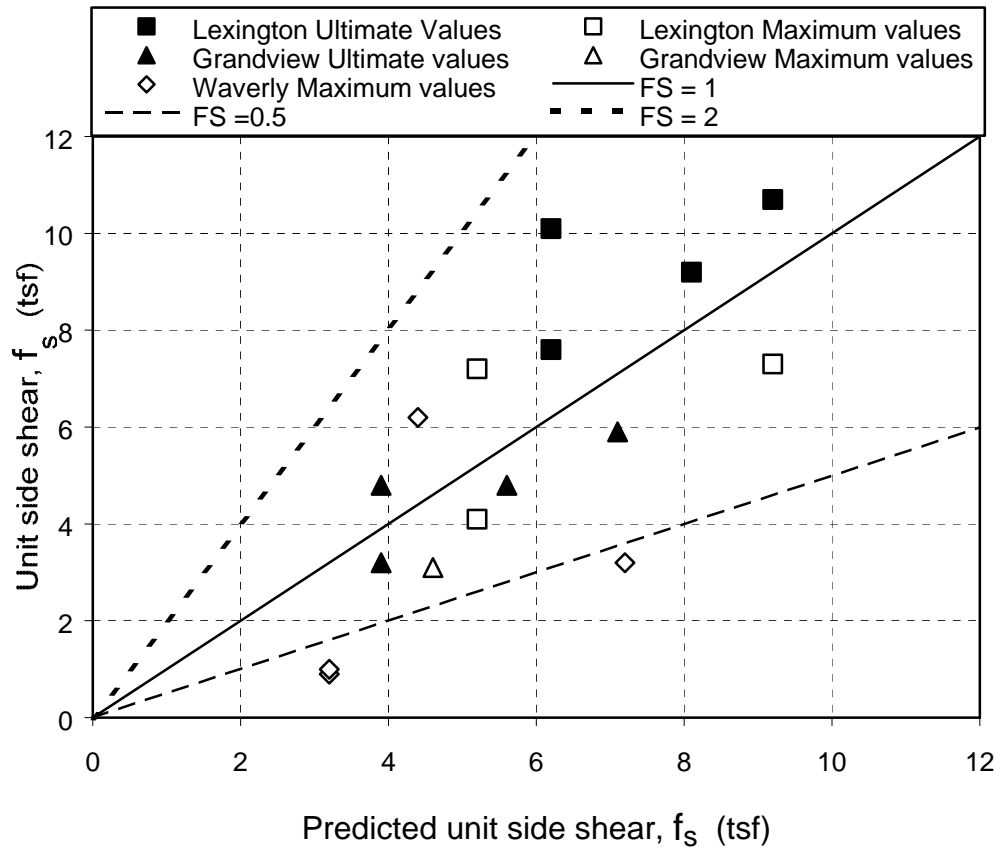
Figure 7.8- Comparison of measured and predicted unit side shear value using the Rowe and Armitage (1987) method.
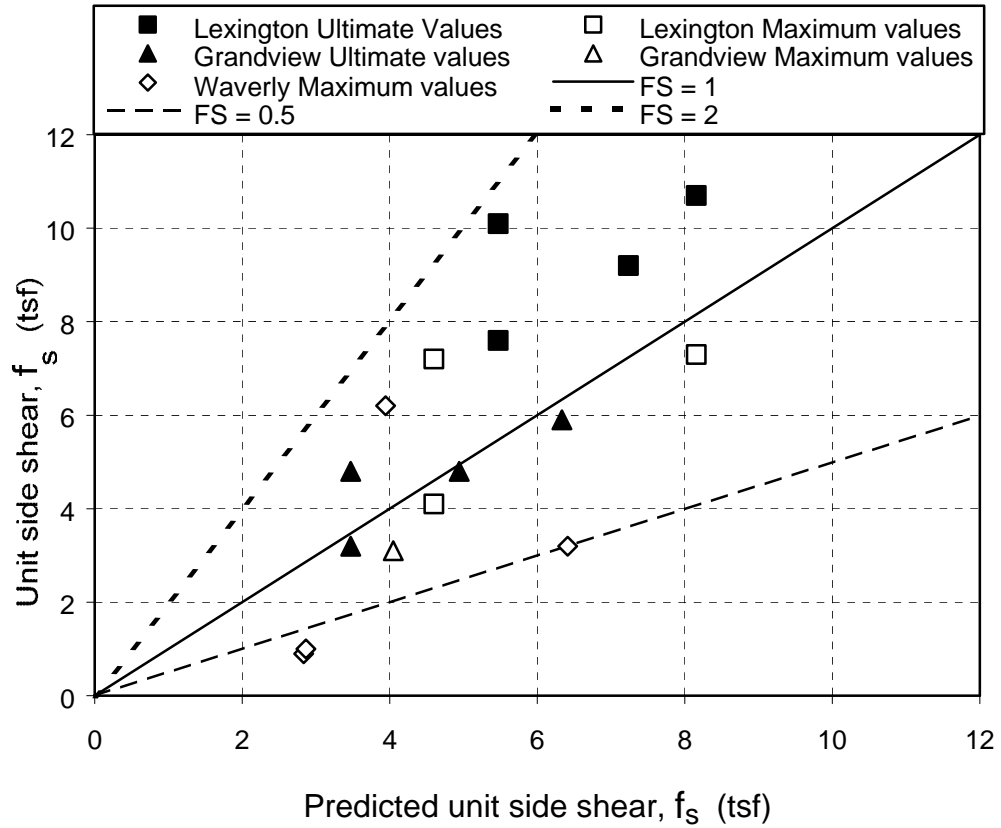
Figure 7.9- Comparison of measured and predicted unit side shear value using the modified Rowe and Armitage (1987) method.

CHAPTER EIGHT
SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

8.1 Summary

The focus of this research study has been to evaluate several design methods for predicting the ultimate unit side shear of drilled shafts socketed into weak Pennsylvanian shales. Load tests were performed on four full-scale, instrumented drilled shafts at three sites using the Osterberg load cell.

A literature survey was undertaken to identify a number of available design methods. Empirical and analytical methods for predicting the ultimate side shear capacity of drilled shafts socketed into weak rock were presented in Chapter 2. Empirical methods are generally based on results of full-scale load tests while analytical methods attempt to model the soft rock-drilled shaft interface numerically, often using finite-element solutions.

In Chapter 3, a new test method for full-scale load testing of drilled shafts using the Osterberg cell (O-cell^{TM}) was described. The Osterberg load cell, along with instrumentation such as strain gages and telltales, may be used to determine end bearing and side shear capacities of drilled shafts and piles.

Load testing of two 1.2 meter (4 ft) diameter drilled shafts located in the Missouri River at Lexington, Missouri was presented in Chapter 4. The drilled shafts were socketed 12.3 and 20.3 meters (40 to 66.6 ft) into bedrock at the bridge site, which consists of Pennsylvanian Age shales, siltstones, sandstones, limestones, and scattered coal beds. Due to difficulty with access to the sites by land and to avoid hindering river traffic, the two Osterberg cell load test shafts were located in the river but near the river

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banks.   Both test shafts were impacted by high river levels and scheduling conflicts.

The drilled shafts were tested to maximum loads of 13.3 MN (1,495 tons) and 17.5 MN

(1,968 tons) by Loadtest Inc. in May and June of 1999. Test shaft TS-1A included one

660mm (26 in) Osterberg load cell while test shaft TS-2 included two 660 mm (26 in)

Osterberg load cells. The Osterberg cell load tests where successful in allowing MoDOT

to develop a more economical design for the drilled shafts for the proposed bridge

alignment. Data from the Osterberg cell load test allowed 1.68 m (5.5 ft) diameter rock

sockets at bridge A5664 to be shortened a total of 704.5 m (2311.5 ft) for a net savings of

$1.8 million.

In Chapter 5, the results of an Osterberg cell load test performed on a 1.8 meter (6

ft) diameter drilled shaft as part of the reconstruction of an intersection in the Kansas City

metropolitan area was presented. The drilled shaft was socketed 13 meters (42.7 ft) into

bedrock, which consisted of horizontally bedded layers of limestones and shales known

as the Kansas City Group. The 870 mm (34 in) Osterberg load cell was successfully

loaded to 33.78 MN (3789 tons) on June 4, 2002. Data from the Osterberg load test

would allow 2.3 m (7.5 ft) diameter rock sockets at bridge A6252 to be shortened a total

of 65.2 m (214 ft) for a net savings of $19 thousand.

Load testing of a "production" drilled shaft with a 660 mm (26 in) Osterberg load

cell is presented in Chapter 6 for a proposed bridge across the Missouri River at Waverly,

Missouri.   The production shaft was one of 6 shafts used to construct the footing of Pier

11 in the middle of the Missouri River. The two-meter (6.5 ft) diameter shaft was

socketed 15.1 meters (49.5 ft) into bedrock consisting of Pennsylvanian Age shales,

siltstones, sandstones, limestones, and scattered coal beds. The production shaft was

tested to a maximum load of 22.5 MN (2,525 tons) on September 30, 2002. The

Osterberg cell load test was successful in testing the shaft to twice the design load and

assuring the foundation engineers that the main pier in the river would be safe.

Finally, measured values of unit side shear determined for various strata involved

in the load tests were collectively analyzed to evaluate the suitability of several design

methods. The results of these analyses are presented in Chapter 7, where the measured

values of unit side shear are compared to values predicted by several design methods. In

addition, conclusion are drawn on the appropriateness of the respective design methods

for use with Missouri shales and a modified design method is proposed. This chapter

provides a summary of this thesis, conclusions reached, lessons learned from the work,

and several recommendations for future work.

8.2 Conclusions

A number of conclusions can be drawn from the results of the four load tests and

subsequent analysis of the test results. Analysis of the load test data indicated that the

ultimate unit side shear may be conservatively estimated as 0.3 times the unconfined

compressive strength of the shale. This more than doubles the values predicted by the

method currently used by MoDOT.

The results of a series of four load tests in Missouri Pennsylvanian Age shales

indicate that design methods by Rowe and Armitage (1987) and Kulhawy and Phoon

(1993) produced almost identical results and most closely predicted the measured

ultimate unit side shear in these materials.

Because the Rowe and Armitage (1987) method slightly over-estimates the

ultimate unit side shear for shale with low compressive strengths, a minor modification of

the method is proposed to produce slightly more conservative values. In the proposed

modification, the ultimate unit side shear is predicted as 0.4 times the square root of q_u

rather than 0.45 as recommended by Rowe and Armitage. This would lead to about a 60

per cent increase in the predicted ultimate unit side shear over current methods followed

by MoDOT.

Methods by Horvath and Kenney (1979) and Reese and O'Neill (1988) produced

similar results. However, both methods significantly under-predicted measured values

observed in the load tests by as much as a factor of 2.

O'Neill and Reese (1999) tended to be more conservative than either Horvath and

Kenney or Reese and O'Neill (1988) when the simplified method for smooth sockets was

used. However, the method proposed by O'Neill and Reese (1999) is not intended for use

when the slump of the concrete is less than 175 mm (7in) as was the case for load tests at

the Lexington site.

The use of the Osterberg cell load test method has lead to significant cost savings

in the design of foundations for bridges, increased the confidence level in design methods

used, and has the potential to improve future designs over time. The testing of two full-

scale shafts in the River at Lexington, Missouri cost approximately $0.5 million and

generated a net cost saving of about $1.8 million. At the Grandview site, data from the

Osterberg cell load test would allow the 7.5 feet (2.3 m) diameter rock sockets at bridge

A6252 to be shortened a total of 214 feet (65.2 m) for a cost savings of $214 thousand.

The cost of the shaft excavation and the Osterberg cell load test was $195 thousand for a

net savings of $19 thousand. At the Waverly site, a production shaft was tested during

the construction phase of the project. Although it is difficult to obtain cost savings during construction, with most construction testing being "proof" testing (testing to twice the design load), about 3 feet (0.9 m) of socket was eliminated on all 6 shafts at Pier 11. The shortening of the sockets saved $29 thousand to offset the $70 thousand cost for the Osterberg cell load test. The cost savings at the three test sites indicate the magnitude of cost savings that could be realized in using Osterberg cell load tests on future projects.

8.3 Lessons Learned

A number of lessons were learned in performing and evaluating these load tests. The collapse of test shaft TS-1 at the Lexington site has led to specification changes that require rock sockets to be excavated and the shaft concrete to be placed within 3 days for shafts constructed in shales. Further specification changes require the use of polymer slurry in drilled shafts constructed in shale that cannot be constructed in the dry.

The Osterberg cell load test at test shaft TS-1A at the Lexington site did not occur as planned. No failure was achieved in TS-1A even though the capacity of the Osterberg load cell was not reached. The test was stopped because the applied pressure to the cell exceeded the capacity of the pressure gage and a higher capacity gage was not available on site. Care should be taken in future load tests using the O-cell^{TM} to ensure that adequate pressure gages are available.

Strain gages placed near the O-cells^{TM} did not function correctly at all three sites due to the zone of influence caused by the non-rigid bearing plates. In future tests using the Osterberg load cell, strain gages should be located no closer than one diameter, and preferably two diameters from the O-cell^{TM}.

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Problems were also experienced with clogging of the concrete tremie pipes for pipes with diameters less than 30 cm (12 in) at two of the sites. In the future, significant effort should be made to maintain a minimum concrete slump of 203mm (8 in) at the time of concrete placement. The holes in the O-cell^{TM} bearing plates should also be enlarged to allow the use of 30 cm (12 in) tremie pipe, particularly for deep shafts.

8.4 Recommendations for Future Work

Several recommendations can be made based on experiences resulting from this research. Due to the limited amount of data for the ultimate side shear capacities of drilled shafts in Missouri shales, further load testing is recommended to expand the database of measured unit side shear values. For additional tests, every effort should be made to load shafts so that side shear is fully mobilized to allow direct evaluation of current design methods.

Due to the difficulty present in accurately testing shales with low strength, the unconfined compression strength, q_u should be replaced by strengths determined from confined triaxial tests (i.e. Q or R tests). In addition, extreme care should be taken in evaluating laboratory strength parameters for all load test sites.

Other methods for predicting the ultimate unit side shear should be evaluated that account for the roughness of the socket and other parameters that may affect the capacity of drilled shafts. One such example is the computer program (ROCKET 95) developed by Seidel and Haberfield (1995). Another method is using borehole shear testing to determine the ultimate unit side shear. Particular attention should also be paid to emerging methods for accurately providing a caliper log of the excavated shaft.

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An evaluation of side shear load capacities for different diameter shafts should be performed and an adjustment developed to account for the diameter of the shaft. Very little testing has been done in this area although expanding cavity theory suggests that there is an effect of diameter on load transfer (Hassan and O'Neill 1997). Baycan (1996) used a computer program (ROCKET 95) developed by Seidel and Haberfield (1995) and found diameter has a significant effect on unit side shear. Having direct knowledge of the relation between capacity and diameter would allow for the testing of smaller diameter shafts, which would lead to substantial cost savings by reducing the costs of constructing the test shafts in addition to reducing the costs of the O-cell tests because smaller cells could be used. This in turn, may lead to more tests being performed because of the reduced costs of each test.

The relation between conventional top-down load testing and Osterberg cell load tests should be investigated. Although many investigators believe that there is little difference in side shear capacities attributed to loading direction, very little full-scale testing exists. Finite element analyses performed by Shi (2002) indicate that, as the modulus of the rock increased, the difference in side shear capacities for top-down and O-cell loading increased with the Osterberg cell load test method becoming more conservative.

Finally, current design methods should be periodically re-evaluated to determine if the predicted ultimate unit side shear can be improved based on new data from additional load tests.

APPENDIX A
DETAILED DATA FOR LOAD TESTS AND UNCONFINED COMPRESSIVE
STRENGTH OF NX CORES AT LEXINGTON SITE

Table A.1
Summary of dimensions, elevations, and shaft properties (TS-1A)

Shaft:

Average Shaft Diameter (EL 175.93 m to 167.27 m)	=	1111 mm	43.75 in
O-Cell^{TM}: 8037-11	=	660 mm	26 in
Length of side shear above break at base of O-cell^{TM}	=	5.42 m	17.8 ft
Length of side shear below break at base of O-cell^{TM}	=	3.24 m	10.6 ft
Shaft side shear area above O-cell^{TM} base	=	19.9 m^2	214.4 ft^2
Shaft side shear area below break at base of O-cell^{TM}	=	11.9 m^2	128.2 ft^2
Shaft base area	=	1.08 m^2	11.6 ft^2
Bouyant weight of shaft above base of O-cell^{TM}	=	0.11 MN	25 kips
Estimated shaft stiffness	=	25,506 MN	5,734,159 kips
(EL 175.93 m to 170.51 m)			
Estimated shaft stiffness	=	25,506 MN	5,734,159 kips
(EL 170.51 m to 167.27 m)			
Elevation of Water Table	=	208.24 m	683.2 ft
Elevation of Mud line	=	207.63 m	681.2 ft
Elevation top of shaft concrete	=	175.93 m	577.2 ft
Elevation of base of O-cell^{TM}	=	170.51 m	559.4 ft
Elevation of shaft tip	=	167.27 m	548.8 ft

Casing:

Elevation of top of permanent casing (1220 mm O.D.)	=	210.68 m	691.2 ft
Elevation of bottom of permanent casing	=	179.59 m	589.2 ft

Compression Sections:

EL. of top of telltale used for shaft compression above cell		175.07 m	574.4 ft
EL. of bottom of telltale used for shaft compression above cell		170.87 m	560.6 ft

Strain Gages:

Elevation of strain gage level 4	=	174.51 m	572.5 ft
Elevation of strain gage level 3	=	173.01 m	567.6 ft
Elevation of strain gage level 2	=	172.01 m	564.3 ft
Elevation of strain gage level 1	=	169.56 m	556.3 ft

Miscellaneous:

Top Plate Diameter	=	815 mm	32.1 in
Bottom Plate Diameter	=	915 mm	36.0 in
Carrying frame cross sectional area (2 No. C4 x 7.25)	=	2748 mm^2	4.26 in^2
Unconfined compressive concrete strength	=	28.1 MPa	4075 psi
O-Cell^{TM} LVWDTs @ 0^0, 90^0, and 180^0 with radius	=	432 mm	17 in

Table A.2- Osterberg O-cell^{TM} versus top and bottom plate movement for load increments 1L-0 to 1U-7 (TS-1A).

Load Test Incre.	O-cell Loads		Upwrd Mvment		Creep	Bottom of Cell		Creep	Dnwrd Mvment
	Gross (MN)	Net (MN)	2 min (mm)	4 min (mm)	2-4 Min (mm)	2 min (mm)	4 min (mm)	2-4 Min (mm)	4 min (mm)
1L-0	0	0	0	0	0	0	0	0	0
1L-1	0.62	0.51	0	0.01	0.01	0.04	0.03	-0.01	-0.03
1L-2	1.11	1	0.02	0.02	0	0.04	0.04	0	-0.04
1L-3	1.58	1.47	0.03	0.02	-0.01	0.07	0.08	0.01	-0.08
1L-4	2.08	1.97	0.1	0.1	0	0.59	0.61	0.02	-0.61
1L-5	2.56	2.45	0.22	0.25	0.03	1.04	1.02	-0.02	-1.02
1L-6	3.04	2.93	0.35	0.36	0.01	1.25	1.25	0	-1.25
1L-7	3.53	3.42	0.44	0.46	0.02	1.42	1.5	0.08	-1.5
1L-8	4.03	3.92	0.53	0.55	0.02	1.67	1.68	0.01	-1.68
1L-9	4.5	4.39	0.62	0.64	0.02	1.88	1.88	0	-1.88
1L-10	4.98	4.87	0.72	0.73	0.01	2.11	2.09	-0.02	-2.09
1L-11	5.46	5.35	0.81	0.83	0.02	2.31	2.34	0.03	-2.34
1L-12	5.93	5.82	0.9	0.95	0.05	2.47	2.54	0.07	-2.54
1L-13	6.43	6.32	1.07	1.11	0.04	2.64	2.66	0.02	-2.66
1L-14	6.9	6.79	1.22	1.24	0.02	2.84	2.85	0.01	-2.85
1L-15	7.38	7.27	1.32	1.34	0.02	3.18	3.27	0.09	-3.27
1L-16	7.85	7.74	1.43	1.45	0.02	3.4	3.43	0.03	-3.43
1L-17	8.35	8.24	1.55	1.57	0.02	3.65	3.72	0.07	-3.72
1L-18	8.82	8.71	1.67	1.69	0.02	3.89	3.93	0.04	-3.93
1L-19	9.28	9.17	1.78	1.79	0.01	4.25	4.27	0.02	-4.27
1L-20	9.74	9.63	1.91	1.91	0	4.58	4.61	0.03	-4.61
1L-21	10.26	10.15	2.02	2.04	0.02	4.92	5.02	0.1	-5.02
1L-22	10.73	10.62	2.15	2.18	0.03	5.26	5.33	0.07	-5.33
1L-23	11.18	11.07	2.3	2.33	0.03	5.59	5.69	0.1	-5.69
1L-24	11.7	11.59	2.45	2.47	0.02	5.91	6.07	0.16	-6.07
1L-25	12.17	12.06	2.6	2.62	0.02	6.39	6.51	0.12	-6.51
1L-26	12.67	12.56	2.75	2.77	0.02	6.73	6.92	0.19	-6.92
1L-27	13.17	13.06	2.94	2.98	0.04	7.32	7.44	0.12	-7.44
1L-28	13.57	13.46	3.1	3.14	0.04	7.7	7.7	0	-7.7
1L-29	14.09	13.98	3.3	3.35	0.05	8.19	8.33	0.14	-8.33
1L-30	14.58	14.47	3.5	3.53	0.03	8.69	8.78	0.09	-8.78
1L-31	15.09	14.98	3.7	3.74	0.04	9.34	9.49	0.15	-9.49
1L-32	15.57	15.46	3.9	3.94	0.04	10.02	10.31	0.29	-10.31
1L-33	16.04	15.93	4.16	4.23	0.07	10.69	10.78	0.09	-10.78
1L-34	16.59	16.48	4.58	4.66	0.08	11.58	11.66	0.08	-11.66
1L-35	17.05	16.94	4.94	5.06	0.12	12.48	12.63	0.15	-12.63
1L-36	17.5	17.39	5.35	5.47	0.12	13.35	13.6	0.25	-13.6
1U-1	16.03	15.92		5.74			13.99		-13.99
1U-2	13.4	13.29		5.57			14.08		-14.08
1U-3	9.93	9.82		5.19			14.14		-14.14
1U-4	6.85	6.74		4.61			13.77		-13.77
1U-5	3.45	3.34		3.77			13.23		-13.23
1U-6	1.81	1.7		3.06			11.47		-11.47
1U-7	0.06	0		2.45			10.74		-10.74

Table A.3- Strain gage data, Lexington, Missouri test site (TS-1A).

Load Test Incre.	O-cell Loads Gross (MN)	Net (MN)	Top of Conc Avg. Load (MN)	Level 4 Avg. Load (MN)	Level 3 Avg. Load (MN)	Level 2 Avg. Load (MN)	O-cell Net (MN)	Level 1 Avg. Load (MN)
Elev (m)			175.93	174.51	173	172	170.5	169.6
1L-0	0	0	0	0	0	0	0	0
1L-1	0.62	0.51	0	0	0.02	0.05	0.51	0.06
1L-2	1.11	1	0	0.01	0.06	0.11	1	0.14
1L-3	1.58	1.47	0	0.03	0.1	0.2	1.47	0.25
1L-4	2.08	1.97	0	0.12	0.36	0.75	1.97	0.76
1L-5	2.56	2.45	0	0.21	0.61	1.24	2.45	1.31
1L-6	3.04	2.93	0	0.3	0.82	1.64	2.93	1.75
1L-7	3.53	3.42	0	0.35	1.05	2.03	3.42	2.2
1L-8	4.03	3.92	0	0.43	1.24	2.37	3.92	2.56
1L-9	4.5	4.39	0	0.49	1.42	2.7	4.39	2.9
1L-10	4.98	4.87	0	0.58	1.61	3.05	4.87	3.26
1L-11	5.46	5.35	0	0.63	1.82	3.43	5.35	3.65
1L-12	5.93	5.82	0	0.69	2	3.76	5.82	4
1L-13	6.43	6.32	0	0.75	2.2	4.14	6.32	4.39
1L-14	6.9	6.79	0	0.8	2.39	4.51	6.79	4.76
1L-15	7.38	7.27	0	0.86	2.59	4.88	7.27	5.14
1L-16	7.85	7.74	0	0.91	2.78	5.24	7.74	5.52
1L-17	8.35	8.24	0	0.96	2.98	5.61	8.24	5.91
1L-18	8.82	8.71	0	1	3.17	5.98	8.71	6.29
1L-19	9.28	9.17	0	1.07	3.35	6.31	9.17	6.62
1L-20	9.74	9.63	0	0.79	3.54	6.71	9.63	7.03
1L-21	10.26	10.15	0	1.16	3.75	7.1	10.15	7.48
1L-22	10.73	10.62	0	1.22	3.95	7.48	10.62	7.89
1L-23	11.18	11.07	0	1.22	4.15	7.87	11.07	8.3
1L-24	11.7	11.59	0	1.32	4.35	8.29	11.59	8.74
1L-25	12.17	12.06	0	1.37	4.57	8.71	12.06	9.19
1L-26	12.67	12.56	0	1.43	4.78	9.13	12.56	9.62
1L-27	13.17	13.06	0	1.47	5	9.52	13.06	10.07
1L-28	13.57	13.46	0	1.52	5.18	9.91	13.46	10.47
1L-29	14.09	13.98	0	1.56	5.39	10.34	13.98	10.97
1L-30	14.58	14.47	0	1.61	5.59	10.76	14.47	11.45
1L-31	15.09	14.98	0	1.66	5.81	11.24	14.98	12.03
1L-32	15.57	15.46	0	1.7	6.01	11.69	15.46	12.55
1L-33	16.04	15.93	0	1.76	6.22	12.15	15.93	13.13
1L-34	16.59	16.48	0	1.81	6.48	12.79	16.48	13.9
1L-35	17.05	16.94	0	1.87	6.75	13.37	16.94	14.63
1L-36	17.5	17.39	0	1.91	6.97	13.88	17.39	15.33
1U-1	16.03	15.92	0	1.94	7.13	14.25	15.92	15.98
1U-2	13.4	13.29	0	21.74	6.74	13.54	13.29	15.17
1U-3	9.93	9.82	0	21.56	6.07	12.18	9.82	13.55
1U-4	6.85	6.74	0	21.32	5.12	10.22	6.74	11.22
1U-5	3.45	3.34	0	21.1	4.04	8.06	3.34	8.65
1U-6	1.81	1.7	0	20.75	2.4	4.77	1.7	5.01
1U-7	0.06	0	0	20.72	1.88	3.7	-0.05	3.67

Table A.4- Unit side shear data, Lexington, Missouri test site (TS-1A).

Load Test Incre	O-cell Loads		Upwrd Mvmnt	Bottom of Cell	Level 1 Avg.	SG-1 to O-cell	Level 2 Avg.	O-cell to SG-2	Level 3 Avg.	SG-2 to SG-3	Level 4 Avg.	SG-3 to SG-4	SG-4 To TOS
	Gross (MN)	Net (MN)	4 min (mm)	4 min (mm)	Load (MN)	O-cell (kPa)	Load (MN)	SG-2 (kPa)	Load (MN)	SG-3 (kPa)	Load (MN)	SG-4 (kPa)	TOS (kPa)
Elev (m)						170.5-169.6		170.5-172		172.0-173		173-174.5	174.5-175.9
1L-0	0	0	0	0	0		0		0		0		
1L-1	0.62	0.51	0.01	0.03	0.06	143	0.05	88	0.02	9	0.00	4	0
1L-2	1.11	1	0.02	0.04	0.14	274	0.11	170	0.06	14	0.01	10	2
1L-3	1.58	1.47	0.02	0.08	0.25	389	0.2	243	0.10	29	0.03	13	6
1L-4	2.08	1.97	0.1	0.61	0.76	386	0.75	233	0.36	112	0.12	46	25
1L-5	2.56	2.45	0.25	1.02	1.31	363	1.24	231	0.61	181	0.21	77	43
1L-6	3.04	2.93	0.36	1.25	1.75	376	1.64	247	0.82	235	0.30	99	61
1L-7	3.53	3.42	0.46	1.5	2.2	389	2.03	266	1.05	281	0.35	134	72
1L-8	4.03	3.92	0.55	1.68	2.56	434	2.37	296	1.24	324	0.43	155	88
1L-9	4.5	4.39	0.64	1.88	2.9	475	2.7	323	1.42	367	0.49	178	100
1L-10	4.98	4.87	0.73	2.09	3.26	513	3.05	348	1.61	413	0.58	197	119
1L-11	5.46	5.35	0.83	2.34	3.65	542	3.43	367	1.82	462	0.63	228	129
1L-12	5.93	5.82	0.95	2.54	4	580	3.76	394	2.00	505	0.69	251	141
1L-13	6.43	6.32	1.11	2.66	4.39	615	4.14	417	2.20	557	0.75	277	154
1L-14	6.9	6.79	1.24	2.85	4.76	647	4.51	436	2.39	608	0.80	304	164
1L-15	7.38	7.27	1.34	3.27	5.14	679	4.88	457	2.59	657	0.86	331	176
1L-16	7.85	7.74	1.45	3.43	5.52	708	5.24	478	2.78	706	0.91	358	186
1L-17	8.35	8.24	1.57	3.72	5.91	743	5.61	503	2.98	755	0.96	386	197
1L-18	8.82	8.71	1.69	3.93	6.29	771	5.98	522	3.17	806	1.00	415	205
1L-19	9.28	9.17	1.79	4.27	6.62	813	6.31	547	3.35	849	1.07	436	219
1L-20	9.74	9.63	1.91	4.61	7.03	829	6.71	559	3.54	910	1.12	463	230
1L-21	10.26	10.15	2.04	5.02	7.48	851	7.1	583	3.75	961	1.16	495	238
1L-22	10.73	10.62	2.18	5.33	7.89	870	7.48	601	3.95	1013	1.22	522	250
1L-23	11.18	11.07	2.33	5.69	8.3	883	7.87	612	4.15	1067	1.22	560	250
1L-24	11.7	11.59	2.47	6.07	8.74	909	8.29	631	4.35	1130	1.32	580	271
1L-25	12.17	12.06	2.62	6.51	9.19	915	8.71	641	4.57	1188	1.37	612	281
1L-26	12.67	12.56	2.77	6.92	9.62	937	9.13	656	4.78	1248	1.43	641	293
1L-27	13.17	13.06	2.98	7.44	10.07	953	9.52	677	5.00	1297	1.47	675	301
1L-28	13.57	13.46	3.14	7.7	10.47	953	9.91	679	5.18	1357	1.52	700	312
1L-29	14.09	13.98	3.35	8.33	10.97	960	10.34	696	5.39	1420	1.56	733	320
1L-30	14.58	14.47	3.53	8.78	11.45	963	10.76	710	5.59	1483	1.61	761	330
1L-31	15.09	14.98	3.74	9.49	12.03	940	11.24	715	5.81	1558	1.66	794	340
1L-32	15.57	15.46	3.94	10.31	12.55	928	11.69	721	6.01	1630	1.70	824	348
1L-33	16.04	15.93	4.23	10.78	13.13	893	12.15	723	6.22	1701	1.76	853	361
1L-34	16.59	16.48	4.66	11.66	13.9	822	12.79	706	6.48	1810	1.81	893	371
1L-35	17.05	16.94	5.06	12.63	14.63	736	13.37	683	6.75	1899	1.87	933	383
1L-36	17.5	17.39	5.47	13.6	15.33	657	13.88	671	6.97	1983	1.91	968	391

Summary of dimensions, elevations, and shaft properties (TS-2)

Shaft:

Average Shaft Diameter (EL 184.4 m to 180.17 m)	=	1182 mm	46.5 in
Average Shaft Diameter (EL 180.17 m to 176.17 m)	=	1167 mm	46.0 in
Average Shaft Diameter (EL 176.17 m to 174.65 m)	=	1107 mm	43.6 in
Upper cell: 8037-13	=	660 mm	26 in
Bottom cell: 8037-12	=	660 mm	26 in
Length of side shear above break at base of upper cell	=	4.23 m	13.9 ft
Length of side shear between Bottom cell and upper cell	=	4.00 m	13.1 ft
Length of side shear below break at base of Bottom-cell	=	1.52 m	5.0 ft
Shaft side shear area above upper cell base	=	15.70 m^2	169 ft^2
Shaft side shear area between bottom cell and upper cell	=	14.67 m^2	157.9 ft^2
Shaft side shear area below break at base of Bottom-cell	=	5.30 m^2	57.1 ft^2
Shaft base area	=	0.96 m^2	10.4 ft^2
Bouyant weight of shaft above base of upper cell	=	0.07 MN	15.6 kips
Bouyant weight of shaft above base of bottom cell	=	0.13 MN	29.0 kips
Estimated shaft modulus (EL 184.4 m to 180.17 m)	=	28.0 GPa	4057 ksi
Estimated shaft modulus (EL 180.17 m to 176.17 m)	=	28.8 GPa	4173 ksi
Estimated shaft modulus (EL 176.17 m to 174.65 m)	=	26.9 GPa	3904 ksi
Elevation of Water Table	=	Variable	Variable
Elevation of Mud line	=	204.52 m	671.0 ft
Elevation top of shaft concrete	=	184.40 m	605.0 ft
Elevation of base of upper cell	=	180.17 m	591.1 ft
Elevation base of bottom cell	=	176.17 m	578.0 ft
Elevation of shaft tip	=	174.65 m	573.0 ft

Casing:

Elevation of top of inner permanent casing(1090 mm O.D.)	=	210.92 m	692.0 ft
Elevation of bottom of inner permanent casing	=	193.85 m	636.0 ft

Compression Sections:

Elevation of top of level 2 telltale	=	183.89 m	603.3 ft
Elevation of bottom of level 2 telltale	=	180.54 m	592.3 ft
Elevation of top of level 1 telltale	=	179.92 m	590.3 ft
Elevation of bottom of level 1 telltale	=	176.57 m	579.3 ft

Strain Gages:

Elevation of strain gage level 4	=	182.67 m	599.3 ft
Elevation of strain gage level 3	=	181.67 m	596.0 ft
Elevation of strain gage level 2	=	178.67 m	586.2 ft
Elevation of strain gage level 1	=	177.67 m	582.9 ft

Miscellaneous:

Carrying frame cross sectional area (C4 x 7.25)	=	8129 mm^2	12.6 in^2
Unconfined compressive concrete str. (EL. 184.4 to 178.46 m)		28.1 MPa	4070 psi
Unconfined compressive concrete str. (EL. 178.46 to 174.65 m)		33.7 MPa	4885 psi

Table A.6- Osterberg O-cells^{TM} versus top and bottom plate movement for load increments 1L-0 to 1U-5 (TS-2, stage 1).

Load Test Incre.	O-cell Loads Lower cell		Avg. Top of Shaft	Avg. Comp. ECT LEVEL 1 Between cells	Avg. Comp. ECT LEVEL 2 Above Upper cell	Avg. Expnsn Upper cell	Shaft Comp.	Upward Mvment Top of Lower Cell	Avg. Expnsn LVWDT 14985 LVWDT 14986 LVWDT 14987 Lower cell	Dwnwrd Mvment
	Gross (MN)	Net (MN)	4 min (mm)	4 min (mm)	4 min (mm)	4 min (mm)	4 min (mm)	4 min (mm)	4 min (mm)	4 min (mm)
1	2	3	5	6	7	8	9	10	11	12
							6+7-8	5+9		10 - 11
1L-0	0	0	0	0	0	0	0	0	0	0
1L-1	0.87	0.74	-0.1	0.02	0	0	0.02	-0.08	0.12	-0.2
1L-2	1.68	1.55	-0.1	0.07	0	0	0.07	-0.03	0.27	-0.3
1L-3	2.49	2.36	-0.14	0.12	0	-0.01	0.13	-0.01	0.51	-0.52
1L-4	3.3	3.17	-0.1	0.2	0	-0.01	0.21	0.11	1.06	-0.95
1L-5	4.11	3.98	-0.24	0.6	0.01	-0.02	0.63	0.39	3.22	-2.83
1L-6	4.92	4.79	-0.14	0.9	0.01	-0.03	0.94	0.8	5.49	-4.69
1L-7	5.73	5.6	-0.03	1.26	0.02	-0.03	1.31	1.28	8.74	-7.46
1L-8	6.54	6.41	0	1.71	0.02	-0.04	1.77	1.77	12.63	-10.86
1L-9	7.35	7.22	0.03	2.23	0.03	-0.05	2.31	2.34	17.83	-15.49
1L-10	8.16	8.03	-0.07	2.92	0.04	-0.06	3.02	2.95	23.41	-20.46
1L-11	8.97	8.84	-0.14	3.98	0.04	-0.06	4.08	3.94	30.18	-26.24
1L-12	9.78	9.65	-0.27	5.39	0.05	-0.07	5.51	5.24	45.27	-40.03
1L-13	10.59	10.46	-0.41	7.23	0.03	-0.09	7.35	6.94	67.62	-60.68
1U-1	6.54	6.41	-0.44	7.36	0.02	-0.06	7.44	7	68.67	-61.67
1U-2	3.3	3.17	-0.55	7.27	0.02	-0.04	7.33	6.78	67.01	-60.23
1U-3	1.68	1.55	-0.48	7.08	0.02	-0.05	7.15	6.67	65.81	-59.14
1U-4	0.87	0.74	-0.41	6.99	0.01	-0.05	7.05	6.64	64.91	-58.27
1U-5	0.06	-0.07	-0.44	6.82	0.01	-0.04	6.87	6.43	63.41	-56.98

Table A.7-Creep data, Lexington, Missouri test site (TS-2, stage 1).

Load Test Incre.	Q-cell Load Lower Cell Net (MN)	Upward Mvment Top of Lower Cell 2 min (mm)	Upward Mvment Top of Lower Cell 4 min (mm)	Creep 2 to 4 min (mm)	Dwnwrd Mvment Bott of Lower Cell 2 min (mm)	Dwnwrd Mvment Lower Cell 4 min (mm)	Creep 2 to 4 min (mm)
1L-0	0	0	0	0	0	0	0
1L-1	0.74	0	-0.08	-0.08	0	0.2	0.2
1L-2	1.55	0.06	-0.03	-0.09	0.21	0.3	0.09
1L-3	2.36	0.07	-0.01	-0.08	0.41	0.52	0.11
1L-4	3.17	0.14	0.11	-0.03	0.87	0.95	0.08
1L-5	3.98	0.46	0.39	-0.07	2.5	2.83	0.33
1L-6	4.79	0.82	0.8	-0.02	4.15	4.69	0.54
1L-7	5.6	1.13	1.28	0.15	6.67	7.46	0.79
1L-8	6.41	1.63	1.77	0.14	9.71	10.86	1.15
1L-9	7.22	2.24	2.34	0.1	14.11	15.49	1.38
1L-10	8.03	2.69	2.95	0.26	19.05	20.46	1.41
1L-11	8.84	3.66	3.94	0.28	24.34	26.24	1.9
1L-12	9.65	4.62	5.24	0.62	33.69	40.03	6.34
1L-13	10.46	6.68	6.94	0.26	57.34	60.68	3.34
1U-1	6.41	7.03	7			61.67	
1U-2	3.17	6.78	6.78			60.23	
1U-3	1.55	6.63	6.67			59.14	
1U-4	0.74	6.5	6.64			58.27	
1U-5	-0.07	6.48	6.43			56.98	

Table A.8-Load distribution data, Lexington, Missouri test site (TS-2, stage 1).

Load Test Incre.	Bottom O-cell Loads		Top of Conc Avg. Load (MN)	Level 4 Avg. Load (MN)	Level 3 Avg. Load (MN)	Level 2 Avg. Load (MN)	O-cell Net (MN)
	Gross (MN)	Net (MN)					
Elev (m)			184.4	182.67	181.67	178.67	176.17
1L-0	0	0	0	0	0	0	0
1L-1	0.87	0.74	0	0	0	0.18	0.74
1L-2	1.68	1.55	0	0.01	0.02	0.28	1.55
1L-3	2.49	2.36	0	0.01	0.03	0.42	2.36
1L-4	3.3	3.17	0	0.03	0.04	0.52	3.17
1L-5	4.11	3.98	0	0.05	0.11	0.66	3.98
1L-6	4.92	4.79	0	0.07	0.16	0.73	4.79
1L-7	5.73	5.6	0	0.1	0.19	0.94	5.6
1L-8	6.54	6.41	0	0.12	0.24	1.16	6.41
1L-9	7.35	7.22	0	0.15	0.31	1.17	7.22
1L-10	8.16	8.03	0	0.17	0.37	1.24	8.03
1L-11	8.97	8.84	0	0.19	0.41	1.54	8.84
1L-12	9.78	9.65	0	0.23	0.49	1.54	9.65
1L-13	10.59	10.46	0	0.22	0.51	3.95	10.46

Table A.9-Unit side shear data, Lexington, Missouri test site (TS-2, stage 1).

Load Test Incre	Upward Movement 4 min (mm)	Downward Movement 4 min (mm)	O-cell Net Lower Cell (MN)	Level 1 Avg. Load (MN)	O-cell to SG-1 176.17 to 177.67 (kPa)	Level 2 Avg. Load (MN)
1L-0	0	0	0	0	0.00	0
1L-1	-0.08	0.2	0.74	0.08	120.07	0.18
1L-2	-0.03	0.3	1.55	0.17	251.07	0.28
1L-3	-0.01	0.52	2.36	0.27	380.24	0.42
1L-4	0.11	0.95	3.17	0.43	498.49	0.52
1L-5	0.39	2.83	3.98	0.67	602.19	0.66
1L-6	0.8	4.69	4.79	0.78	729.55	0.73
1L-7	1.28	7.46	5.6	0.89	856.90	0.94
1L-8	1.77	10.86	6.41	0.98	987.89	1.16
1L-9	2.34	15.49	7.22	0.96	1138.89	1.17
1L-10	2.95	20.46	8.03	0.82	1311.73	1.24
1L-11	3.94	26.24	8.84	0.52	1513.67	1.54
1L-12	5.24	40.03	9.65	1.6	1464.55	1.54
1L-13	6.94	60.68	10.46	2.35	1475.47	3.95

Load Test Incre	SG-1 to SG-2 177.67 to 178.67 (kPa)	Level 3 Avg. Load (MN)	SG-2 to SG-3 178.67 to 181.67 (kPa)	Level 4 Avg. Load (MN)	SG-3 to SG-4 181.67 to 182.67 (kPa)	SG-4 to TOS 182.67 to 184.4 (kPa)
1L-0	0.00	0.00	0.00	0.00	0.00	0.00
1L-1	-27.29	0.00	16.26	0.00	0.00	0.00
1L-2	-30.02	0.02	23.49	0.01	2.69	1.56
1L-3	-40.93	0.03	35.24	0.01	5.39	1.56
1L-4	-24.56	0.04	43.37	0.03	2.69	4.67
1L-5	2.73	0.11	49.69	0.05	16.17	7.79
1L-6	13.64	0.16	51.50	0.07	24.25	10.90
1L-7	-13.64	0.19	67.76	0.10	24.25	15.57
1L-8	-49.12	0.24	83.12	0.12	32.33	18.69
1L-9	-57.31	0.31	77.70	0.15	43.11	23.36
1L-10	-114.62	0.37	78.60	0.17	53.89	26.48
1L-11	-278.36	0.41	102.09	0.19	59.28	29.59
1L-12	16.37	0.49	94.86	0.23	70.05	35.82
1L-13	-436.64	0.51	310.79	0.22	78.14	34.26

Table A.10- Osterberg O-cells^{TM} versus top and bottom plate movement for load increments 2L-0 to 2U-2 (TS-2, stage 2 & 3).

Load Test Incre.	O-cell Loads					Avg. Top of Shaft	Avg. Comp. ECT LEVEL 1 Between cells	Avg. Comp. ECT LEVEL 2 Above upper Cell	Avg. Expansion Upper Cell LVWDT 14991 14992	Shaft Comp.	Upward Mvment Top of Lower Cell	Avg. Expnsion Lower Cell LVWDT 14985 14986	Dwnwrd Mvment
	Lower Cell		Upper Cell										
				Net**	Net***								
	Gross (MN)	Net* (MN)	Gross (MN)	Up (MN)	Down (MN)	4 min (mm)	4 min (mm)	4 min (mm)	4 min (mm)	4 min (mm)	4 min (mm)	4 min (mm)	4 min (mm)
1	2	3			4	5	6	7	8	9	10	11	12
										6+7-8	5+9		10 - 11
2L-0	0.06	0.00	0.00	0.00	0.00	0.00	6.82	0.00	0.00	6.82	6.82		6.82
2L-1	0.06	0.00	0.85	0.78	0.91	-0.44	6.79	0.01	0.09	6.71	6.27	63.11	-56.84
2L-2	0.06	0.00	1.66	1.59	1.72	-0.48	6.80	0.02	0.17	6.65	6.17	63.02	-56.85
2L-3	0.06	0.00	2.48	2.41	2.54	-0.48	6.80	0.03	0.29	6.54	6.06	62.95	-56.89
2L-4	0.06	0.00	3.29	3.22	3.35	-0.41	6.81	0.11	0.56	6.36	5.95	62.86	-56.91
2L-5	0.06	0.00	4.10	4.03	4.16	-0.44	6.85	0.30	1.19	5.96	5.52	62.74	-57.22
2L-6	0.06	0.00	4.91	4.84	4.97	-0.48	6.88	0.44	1.76	5.56	5.08	62.57	-57.49
2L-7	0.06	0.00	5.72	5.65	5.78	-0.27	6.93	0.57	2.35	5.15	4.88	62.37	-57.49
2L-8	0.06	0.00	6.53	6.46	6.59	-0.27	6.99	0.70	2.96	4.73	4.46	62.13	-57.67
2L-9	0.06	0.00	7.34	7.27	7.40	-0.10	7.06	0.84	3.67	4.23	4.13	61.82	-57.69
2L-10	0.06	0.00	8.16	8.09	8.22	0.10	7.16	0.98	4.64	3.50	3.60	61.46	-57.86
2L-11	0.06	0.00	8.97	8.90	9.03	0.31	7.37	1.14	6.57	1.94	2.25	61.03	-58.78
2L-12	0.06	0.00	9.78	9.71	9.84	0.96	7.78	1.33	9.93	-0.82	0.14	60.58	-60.44
2L-13	0.06	0.00	10.59	10.52	10.65	1.47	8.62	1.56	15.91	-5.73	-4.26	60.00	-64.26
2L-14	1.89	1.76	11.40	11.33	11.33	2.32	12.10	2.08	47.21	-33.03	-30.71	57.37	-88.08
2L-15	3.31	3.18	12.21	12.14	12.14	2.56	13.61	2.75	67.71	-51.35	-48.79	56.75	-105.54
2L-16	4.61	4.48	13.03	12.96	12.96	3.31	15.99	3.22	130.92	-111.71	-108.40	56.25	-164.65
2L-17	5.23	5.10	13.35	13.28	13.28	3.72	16.51	3.56	148.15	-128.08	-124.36	56.05	-180.41
2U-1	3.53	3.40	0.00	0.00	0.00	2.79	15.97	3.17			2.79		2.79
2U-2	3.22	3.09	0.00	0.00	0.00	2.44	15.93	3.14			2.44		2.44

* Net load calculated as Lower O-cell^{TM} load minus weight of shaft above Lower O-cell^{TM} = 0.13 MN.

** Net load calculated as Upper O-cell^{TM} load minus weight of shaft above Upper O-cell^{TM} = 0.07 MN.

*** Net load calculated as Upper O-cell^{TM} load plus weight of shaft between the O-cells^{TM} = 0.06 MN (2L-1 to 2L-13).

*** Net load calculated as Upper O-cell^{TM} load minus weight of shaft above the upper O-cells^{TM} = 0.07 MN (2L-14 to 2L-17).

Table A.11-Creep data, Lexington, Missouri test site (TS-2, stage 2 & 3).

Load Test Incre.	O-cell Load Upper cell Net (MN)	Upward Mvment Top of Upper Cell 2 min (mm)	Upward Mvment Top of Upper Cell 4 min (mm)	Creep 2 to 4 min Stages 2&3 (mm)	Dwnwrd Mvment Bott of Upper Cell 2 min (mm)	Dwnwrd Mvment Bott of Upper Cell 4 min (mm)	Creep 2 to 4 min Stage 2 (mm)
2L-0	0	0	0	0.00	0	0	0
2L-1	0.79	-0.03	0.04	0.07	0.11	0.05	-0.06
2L-2	1.6	-0.1	0.01	0.11	0.27	0.17	-0.1
2L-3	2.42	0.03	0.02	-0.01	0.25	0.26	0.01
2L-4	3.23	0.07	0.17	0.10	0.48	0.4	-0.08
2L-5	4.04	0.37	0.32	-0.05	0.78	0.87	0.09
2L-6	4.85	0.52	0.43	-0.09	1.16	1.33	0.17
2L-7	5.66	0.68	0.76	0.08	1.61	1.59	-0.02
2L-8	6.47	0.89	0.9	0.01	2	2.06	0.06
2L-9	7.29	1.16	1.21	0.05	2.39	2.46	0.07
2L-10	8.1	1.53	1.55	0.02	2.9	3.09	0.19
2L-11	8.91	1.94	1.91	-0.03	4.16	4.66	0.5
2L-12	9.72	2.56	2.75	0.19	6.52	7.18	0.66
2L-13	10.53	3.4	3.49	0.09	10.39	12.42	2.03
2L-14	11.34	4.65	4.87	0.22	38.43	42.34	
2L-15	12.15	5.68	5.78	0.10	57.94	61.93	
2L-16	12.97	6.93	7	0.07	118.21	123.93	
2L-17	13.29	7.51	7.75	0.24	134.69	140.4	

Table A.12-Load distribution data, Lexington, Missouri test site (TS-2, stage 2 & 3).

Load Test Incre.	Top of Conc Avg. Load (MN)	Level 4 Avg. Load (MN)	Level 3 Avg. Load (MN)	Upper-Cell Net Load (MN)	Lower Cell Net Load (MN)
Elevation	184.4	182.67	181.67	180.17	176.17
2L-0	0	0.04	0.13	0	0.06
2L-1	0	0.03	0.16	0.78	0.06
2L-2	0	0.05	0.2	1.59	0.06
2L-3	0	0.08	0.24	2.41	0.06
2L-4	0	0.14	0.42	3.22	0.06
2L-5	0	0.32	0.9	4.03	0.06
2L-6	0	0.45	1.24	4.84	0.06
2L-7	0	0.57	1.57	5.65	0.06
2L-8	0	0.69	1.9	6.46	0.06
2L-9	0	0.79	2.22	7.27	0.06
2L-10	0	0.88	2.51	9.09	0.06
2L-11	0	0.97	2.78	8.9	0.06
2L-12	0	1.06	2.95	9.71	0.06
2L-13	0	1.16	3.19	10.52	0.06
2L-14	0	1.24	3.45	11.33	1.89
2L-15	0	1.36	3.72	12.14	3.3
2L-16	0	1.42	3.92	12.96	4.61
2L-17	0	1.52	4.09	13.28	5.23

Table A.13-Unit side shear data, Lexington, Missouri test site (TS-2, stage 2 & 3).

Load Test Icre.	Upper Cell Upward Mvment Top of Cell	Upper Cell Dwnwrd Mvment	Bottom of Cell Net Load	O-cell to Upper cell	Upper Cell Net Load	Upper cell to SG-3	Level 3 Avg. Load	SG-3 to SG-4	Level 4 Avg. Load	SG-4 to TOS	Upper Cell to TOS
				176.17 to 180.17		180.17 to 181.67		181.67 to 182.67		182.67 to 184.4	180.17 to 184.4
	4 min (mm)	4 min (mm)	176.17 (MN)	(kPa)	180.17 (MN)	(kPa)	181.67 (MN)	(kPa)	182.67 (MN)	184.4 (kPa)	(kPa)
2L-0	0.00	0	0.06	0.00	0	0.00	0.13	24.25	0.04	6.23	0.00
2L-1	0.04	0.05	0.06	49.12	0.78	111.37	0.16	35.03	0.03	4.67	49.68
2L-2	0.01	0.16	0.06	104.38	1.59	249.68	0.2	40.42	0.05	7.79	101.28
2L-3	0.02	0.27	0.06	160.33	2.41	389.78	0.24	43.11	0.08	12.46	153.51
2L-4	0.17	0.39	0.06	215.59	3.22	502.94	0.42	75.44	0.14	21.80	205.10
2L-5	0.33	0.86	0.06	270.85	4.03	562.22	0.9	156.27	0.32	49.84	256.70
2L-6	0.43	1.33	0.06	326.11	4.84	646.64	1.24	212.85	0.45	70.08	308.29
2L-7	0.77	1.58	0.06	381.37	5.65	732.86	1.57	269.43	0.57	88.77	359.88
2L-8	0.90	2.06	0.06	436.64	6.46	819.08	1.9	326.02	0.69	107.46	411.48
2L-9	1.21	2.46	0.06	491.90	7.27	907.10	2.22	385.29	0.79	123.04	463.07
2L-10	1.55	3.09	0.06	548.52	8.1	1004.09	2.51	439.18	0.88	137.05	515.94
2L-11	1.92	4.65	0.06	603.10	8.9	1099.29	2.78	487.68	0.97	151.07	566.89
2L-12	2.76	7.17	0.06	658.37	9.71	1214.25	2.95	509.23	1.06	165.09	618.49
2L-13	3.50	12.41	0.06	713.63	10.52	1316.64	3.19	546.95	1.16	180.66	670.08
2L-14	4.87	42.34	1.89	644.04	11.33	1415.43	3.45	595.45	1.24	193.12	721.68
2L-15	5.78	61.93	3.3	603.10	12.14	1512.42	3.72	635.86	1.36	211.81	773.27
2L-16	7.00	123.92	4.61	569.67	12.96	1623.79	3.92	673.59	1.42	221.15	825.50
2L-17	7.75	140.4	5.23	549.21	13.28	1650.73	4.09	692.45	1.52	236.73	845.88

Table A.14- Unconfined compressive strength of NX rock cores

	Mulky	Lagonda	Bevier C1	Bevier C2	Verdigris	Croweburg	Fleming
	(kPa)	(kPa)	(kPa)	(kPa)	(kPa)	(kPa)	(kPa)
	340	680	5759	1948	386	1540	150
	140	140	5465	2060	724	443	333
	110	720	1320	3005	423	1695	620
	310	4807	2473	1646	1280	600	360
		7520	3412	2330	2440	568	195
		510	1648	7130	477	253	1241
		1381	8105	1579	4482	2760	908
		2060	2097	4108	2290	340	
		720	3282	2213	660	3844	
		1907	2140	1600	1727	1760	
		2440	2101	2870	310	452	
		1141	1020	1996	339	1562	
		780	7006	3168	218	5590	
		949	5788	11550		2620	
		914	7806	311			
		1120	5481	2332			
		860	4500	2855			
		637	1579	1320			
		535	2494				
			2578				
			3650				
			7870				
			2581				
			2662				
			2468				
Mean	225	1570	3811	3001	1212	1716	544
Std. Dev.	117	1775	2210	2565	1245	1552	404

Table A.15- Unconfined compressive strength and SPT data for Mulky and Lagonda
Formations.

Boring	Elev.	Unconfined Compressive Strength and SPT Data					
		Pier 19	Pier 20	Pier 21	Pier 22	Pier 23	Pier 24
	(m)	(kPa)	(kPa)	(kPa)	(kPa)	(kPa)	(kPa)
		Mulky					
F-41	200.7						340
F-42	200.0						140
F-41	199.0						110
F-41	197.7						310
		Lagonda					
F-42	197.2						680
F-39	195.6					140	
F-40	195.4					720	
F-39	194.4					12,255*	
B-14	193.7				50 in 8 cm		
B-13	192.7				4807		
F-42	192.5						25,830*
B-15	192.3				50 in 6 cm		
F-39	191.9					7520	
F-40	191.7					510	
B-13	191.5				1381		
F-39	191.3					2060	
B-14	191.1				100 in 13 cm		
B-15	191.1				720		
B-14	190.3				1907		
B-13	190.1				50 in 8 cm		
F-39	189.8					2440	
B-15	189.7				1141		
F-40	189.4					780	
B-13	189.1				949		
B-15	188.9				50 in 11 cm		
B-14	188.9				914		
F-39	188.3					1120	
F-40	188.1					860	
B-13	187.9				637		
B-15	187.9				535		

* Values not used in calculation of mean

Table A.16- Unconfined compressive strength and SPT data for Bevier C1 Formation.

Boring	Elev.	Unconfined Compressive Strength and SPT Data					
		Pier 19	Pier 20	Pier 21	Pier 22	Pier 23	Pier 24
	(m)	(kPa)	(kPa)	(kPa)	(kPa)	(kPa)	(kPa)
		Bevier C1					
B-15	186.9				5759		
F-39	186.7					5465	
B-14	186.5				50 in 8 cm		
B-15	185.8				50 in 7 cm		
B-13	185.7				1320		
B-12	185.6			100 in 21 cm			
B-13	185.6				50 in 6 cm		
Pier 22	185.0	Scour	Depth				
B-13	185.0				2473		
B-14	185.0				3412		
B-15	184.8				1648		
B-11	184.1			50 in 4 cm			
TS-2	183.8				2140		
B-12	183.7			8105			
B-15	183.7				2097		
B-10	183.5			3282			
B-13	183.2				2101		
TS-2	183.1				1020		
B-11	183.0			7006			
B-15	182.8				50 in 5 cm		
B-12	182.7			5788			
B-14	182.6				7806		
Pier 21	182.1	Scour	Depth				
B-10	181.9			5481			
TS-2	181.9				4500		
B-14	181.9				50 in 6 cm		
B-3	181.4				100 in 8 cm		
B-13	181.3				2494		
B-9			50 in 6 cm				
B-13	181.0				50 in 6 cm		
B-13	180.7				2578		
TS-2	180.7				3650		
B-3	180.6		7870				
B-10	180.6			2581			
B-14	180.6				2662		

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Table A.17- Unconfined compressive strength and SPT data for Bevier C2 Formation.

Boring	Elev.	Unconfined Compressive Strength and SPT Data					
		Pier 19	Pier 20	Pier 21	Pier 22	Pier 23	Pier 24
	(m)	(kPa)	(kPa)	(kPa)	(kPa)	(kPa)	(kPa)
		Bevier C2					
B-12	180.3			100 in 8cm			
B-15	180.3				2468		
B-14	179.9				1948		
B-8	179.8		2060				
B-12	179.8			3005			
B-3	179.6				100 in 13 cm		
B-11	179.5			50 in 6 cm			
F-36	179.3	100 in 10cm					
F-37	179.3	100 in 8 cm					
B-10	179.2			1646			
TS-2	179.2				2330		
B-3	179.0		7130				
B-15	179.0				1579		
B-11	178.5			4108			
B-13	178.4				2213		
B-8	178.2		1600				
B-9	177.9		100 in 9 cm				
F-37	177.9	2870					
B-10	177.9			1996			
B-9	177.7		3168				
B-3	177.6		11,550				
B-12	177.5			311			
B-15	177.5				2332		
B-14	177.4				50 in 10 cm		
B-14	177.1				2855		
B-8	176.9			1320			

Table A.18- Unconfined compressive strength and SPT data for Vedigris Formation.

Boring	Elev.	Unconfined Compressive Strength and SPT Data					
		Pier 19	Pier 20	Pier 21	Pier 22	Pier 23	Pier 24
	(m)	(kPa)	(kPa)	(kPa)	(kPa)	(kPa)	(kPa)
		Verdigris					
B-12	176.4			386			
B-3	176.3				100 in 6 cm		
F-36	176.2	100 in 11 cm					
B-10	176.1			724			
B-14	176.1				423		
B-11	175.8			1280			
B-3	175.4		2440				
B-9	175.3		477				
B-12	175.1			4482			
TS-2	175.1				2290		
F-37	174.9	100 in 18 cm					
B-9	174.8		50 in 8 cm				
B-3	174.7				100 in 11 cm		
B-10	174.4			1727			
TS-2	174.3				310		
B-12	174.2			100 in 14 cm			
B-9	173.9		339				
B-11	173.5			50 in 10 cm			
B-11	173.3			218			

Table A.19- Unconfined compressive strength and SPT data for Croweburg Formation and Fleming Formation.

Boring	Elev.	Unconfined Compressive Strength and SPT Data					
		Pier 19	Pier 20	Pier 21	Pier 22	Pier 23	Pier 24
	(m)	(kPa)	(kPa)	(kPa)	(kPa)	(kPa)	(kPa)
		Croweburg					
F-36	173.1	100 in 14 cm					
B-3	173.1				100 in 2 cm		
B-8	172.8		1540				
B-10	172.8			443			
B-3	172.6		16460*				
B-9	172.0		1695				
TS-2	171.9				600		
B-3	171.6				100 in 15 cm		
B-12	171.1			568			
B-10	171.0			253			
B-8	170.8		2760				
TS-2	170.7				340		
B-10	170.6			3844			
B-3	170.4		1760				
F-37	169.9	170*					
B-12	169.6			452			
B-12	169.5			100 in 1 cm			
B-11	169.4			1562			
B-8	169.1		5590				
B-3	168.9		2620				
		Fleming					
TS-2	168.7				150		
B-9	168.6		333				
B-3	168.3				100 in 6 cm		
TS-2	168.0				620		
B-8	167.9		360				
TS-2	167.2				195		
B-9	166.9		1241				
B-9	165.4		908				

* Values not used in calculation of mean

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APPENDIX B
DETAILED DATA FOR LOAD TESTS AND UNCONFINED COMPRESSIVE
STRENGTH OF NX CORES AT GRANDVIEW SITE

Summary of Dimensions, Elevations, and Shaft Properties

Shaft:

Average Shaft Diameter (EL 934.0 ft to 905.0 ft)	=	1976 mm	77.8 in
Average Shaft Diameter (EL 905.0 ft to 893.4 ft)	=	1938 mm	76.3 in
O-cell^{TM}: 2173-3	=	870 mm	34 in
Length of side shear above break at base of O-cell^{TM}	=	8.64 m	29.0 ft
Length of side shear below break at base of O-cell^{TM}	=	3.54 m	11.6 ft
Shaft side shear area above O-cell^{TM} base	=	53.64 m^2	590.7 ft^2
Shaft side shear area below break at base of O-cell^{TM}	=	21.55 m^2	231.7 ft^2
Shaft base area	=	2.95 m^2	31.8 ft^2
Bouyant weight of shaft above base of O-cell^{TM}	=	0.53 MN	118 kips
Estimated shaft stiffness	=	93,800 MN	21,100,000 kips
(EL 934.0 ft to 905.0 ft)			
Estimated shaft stiffness	=	90,300 MN	20,300,000 kips
(EL 905.0 ft to 893.4 ft)			
Elevation of Water Table	=	279.61 m	917.4 ft
Elevation of Mud line	=	287.23 m	942.4 ft
Elevation top of shaft concrete	=	284.67 m	934.0 ft
Elevation of base of O-cell^{TM}	=	275.84 m	905.0 ft
Elevation of shaft tip	=	272.29 m	893.4 ft

Casing:

Elevation of top of temporary casing (2134 mm O.D.)	=	287.53 m	943.4 ft
Elevation of bottom of temporary casing (84 in O.D.)	=	285.32 m	936.1 ft

Compression Sections:

EL. of top of telltale used for upper shaft compression	=	284.67 m	934.0 ft
EL. of bottom of telltale used upper shaft compression	=	276.23 m	906.3 ft
EL. of top of telltale used lower shaft compression	=	275.79 m	904.8 ft
EL. of bottom of telltale used lower shaft compression	=	272.36 m	893.6 ft

Strain Gages:

Elevation of strain gage level 6	=	282.54 m	927.0 ft
Elevation of strain gage level 5	=	280.71 m	921.0 ft
Elevation of strain gage level 4	=	279.34 m	916.5 ft
Elevation of strain gage level 3	=	277.81 m	911.5 ft
Elevation of strain gage level 2	=	274.62 m	901.0 ft
Elevation of strain gage level 1	=	273.70 m	898.0 ft

Miscellaneous:

Top Plate Diameter	=	1676 mm	66 in
Bottom Plate Diameter	=	1676 mm	66 in
Frame cross sectional area (2 No. C4x7.25)	=	2748 m^2	4.26 in^2
Rebar cage diameter	=	1676 mm	66 in
Spiral size (60 in spacing)	=	M 16	# 5
Unconfined compressive concrete strength	=	41.4 MPa	6000 psi
O-cell^{TM} LVWDTs @ 0^0, 90^0, and 180^0 with radius	=	500 mm	20 in

Table B.2- Osterberg O-cell^{TM} versus top and bottom plate movement for load increments 1L-0 to 1L-21.

Load Test Incre	Hold Time Min	O-cell Loads Gross (MN)	Gross (tons)	Net (MN)	Net (tons)	Top of Shaft (in)	Upper Compre (in)	Upward Mvemnt Top Plate (mm)	(in)	O-cell expansion (mm)	(in)	Dnwrd Mvment (mm)	(in)
						A	B	A + B		C		A + B - C	
1L-0		0.00	0	0	0	0	0	0.00	0	0	0	0	0
1L-1	1	1.72	193.5	1.19	134.5	0.004	0.002	0.15	0.006	0.33	0.013	-0.18	-0.007
1L-1	2	1.63	183.5	1.10	124.5	0.004	0.002	0.15	0.006	0.33	0.013	-0.18	-0.007
1L-1	4	1.72	193.6	1.19	134.6	0.004	0.002	0.15	0.006	0.33	0.013	-0.18	-0.007
1L-2	1	3.26	367	2.73	308	0.005	0.002	0.18	0.007	0.36	0.014	-0.18	-0.007
1L-2	2	3.26	367	2.73	308	0.005	0.002	0.18	0.007	0.36	0.014	-0.18	-0.007
IL-2	4	3.26	367	2.73	308	0.005	0.002	0.18	0.007	0.36	0.014	-0.18	-0.007
1L-3	1	4.90	550.5	4.37	491.5	0.007	0.003	0.25	0.010	0.43	0.017	-0.18	-0.007
1L-3	2	4.90	550.5	4.37	491.5	0.007	0.003	0.25	0.010	0.43	0.017	-0.18	-0.007
1L-3	4	4.90	550.5	4.37	491.5	0.007	0.003	0.25	0.010	0.43	0.017	-0.18	-0.007
1L-4	1	6.53	734	6.00	675	0.007	0.004	0.28	0.011	0.46	0.018	-0.18	-0.007
1L-4	2	6.53	734	6.00	675	0.008	0.004	0.30	0.012	0.46	0.018	-0.15	-0.006
1L-4	4	6.53	734	6.00	675	0.008	0.004	0.30	0.012	0.48	0.019	-0.18	-0.007
1L-5	1	8.16	917.5	7.63	858.5	0.010	0.005	0.38	0.015	0.56	0.022	-0.18	-0.007
1L-5	2	8.16	917.5	7.63	858.5	0.010	0.006	0.41	0.016	0.53	0.021	-0.13	-0.005
1L-5	4	8.16	917.5	7.63	858.5	0.012	0.006	0.46	0.018	0.56	0.022	-0.10	-0.004
1L-6	1	9.79	1101	9.26	1042	0.014	0.007	0.53	0.021	0.69	0.027	-0.15	-0.006
1L-6	2	9.79	1101	9.26	1042	0.015	0.006	0.53	0.021	0.64	0.025	-0.10	-0.004
1L-6	4	9.79	1101	9.26	1042	0.015	0.007	0.56	0.022	0.66	0.026	-0.10	-0.004
1L-7	1	11.43	1284.5	10.90	1225.5	0.020	0.008	0.71	0.028	1.02	0.04	-0.30	-0.012
1L-7	2	11.43	1284.5	10.90	1225.5	0.019	0.008	0.69	0.027	1.07	0.042	-0.38	-0.015
1L-7	4	11.43	1284.5	10.90	1225.5	0.020	0.009	0.74	0.029	1.07	0.042	-0.33	-0.013
1L-8	1	13.06	1468	12.53	1409	0.024	0.010	0.86	0.034	1.17	0.046	-0.30	-0.012
1L-8	2	13.06	1468	12.53	1409	0.025	0.010	0.89	0.035	1.19	0.047	-0.30	-0.012
1L-8	4	13.06	1468	12.53	1409	0.026	0.010	0.91	0.036	1.24	0.049	-0.33	-0.013
1L-9	1	14.69	1651.5	14.16	1592.5	0.036	0.011	1.19	0.047	1.63	0.064	-0.43	-0.017
1L-9	2	14.69	1651.5	14.16	1592.5	0.036	0.012	1.22	0.048	1.68	0.066	-0.46	-0.018
1L-9	4	14.69	1651.5	14.16	1592.5	0.036	0.012	1.22	0.048	1.73	0.068	-0.51	-0.020
1L-10	1	16.32	1835	15.79	1776	0.044	0.013	1.45	0.057	1.91	0.075	-0.46	-0.018
1L-10	2	16.32	1835	15.79	1776	0.046	0.013	1.50	0.059	2.29	0.09	-0.79	-0.031
1L-10	4	16.32	1835	15.79	1776	0.050	0.013	1.60	0.063	2.34	0.092	-0.74	-0.029
1L-11	1	17.96	2018.5	17.43	1959.5	0.062	0.015	1.96	0.077	2.62	0.103	-0.66	-0.026
1L-11	2	17.96	2018.5	17.43	1959.5	0.066	0.015	2.06	0.081	3.07	0.121	-1.02	-0.040
1L-11	4	17.96	2018.5	17.43	1959.5	0.071	0.015	2.18	0.086	3.15	0.124	-0.97	-0.038
1L-12	1	19.59	2202	19.06	2143	0.085	0.017	2.59	0.102	3.71	0.146	-1.12	-0.044
1L-12	2	19.59	2202	19.06	2143	0.090	0.017	2.72	0.107	3.81	0.15	-1.09	-0.043
1L-12	4	19.59	2202	19.06	2143	0.096	0.017	2.87	0.113	3.94	0.155	-1.07	-0.042
1L-13	1	21.22	2385.5	20.69	2326.5	0.113	0.019	3.35	0.132	4.09	0.161	-0.74	-0.029
1L-13	2	21.22	2385.5	20.69	2326.5	0.121	0.019	3.56	0.140	4.72	0.186	-1.17	-0.046
1L-13	4	21.22	2385.5	20.69	2326.5	0.127	0.019	3.71	0.146	5.26	0.207	-1.55	-0.061

Table B.2- Continued

Load	Hold	O-cell Loads				Top of	Upper	Upward Mvemnt		O-cell		Dnwrd Mvment	
Test	Time	Gross	Gross	Net	Net	Shaft	Compre	Top Plate		expansion			
Incre	Min	(MN)	(tons)	(MN)	(tons)	(in)	(in)	(mm)	(in)	(mm)	(in)	(mm)	(in)
						A	B	A + B		C		A + B - C	
1L-14	1	22.85	2568.5	22.32	2509.5	0.157	0.021	4.52	0.178	6.17	0.243	-1.65	-0.065
1L-14	2	22.85	2568.5	22.32	2509.5	0.162	0.021	4.65	0.183	6.27	0.247	-1.63	-0.064
1L-14	4	22.86	2569.5	22.33	2510.5	0.171	0.021	4.88	0.192	6.43	0.253	-1.55	-0.061
1L-15	1	24.48	2752	23.95	2693	0.197	0.023	5.59	0.220	7.32	0.288	-1.73	-0.068
1L-15	2	24.48	2752	23.95	2693	0.207	0.023	5.84	0.230	7.87	0.31	-2.03	-0.080
1L-15	4	24.48	2752	23.95	2693	0.220	0.023	6.17	0.243	8.08	0.318	-1.91	-0.075
1L-16	1	26.11	2935.5	25.58	2876.5	0.250	0.025	6.99	0.275	9.12	0.359	-2.13	-0.084
1L-16	2	26.11	2935.5	25.58	2876.5	0.267	0.025	7.42	0.292	9.65	0.38	-2.24	-0.088
1L-16	4	26.11	2935.5	25.58	2876.5	0.285	0.025	7.87	0.310	10.36	0.408	-2.49	-0.098
1L-17	1	27.75	3119	27.22	3060	0.321	0.026	8.81	0.347	11.43	0.45	-2.62	-0.103
1L-17	2	27.75	3119	27.22	3060	0.329	0.026	9.02	0.355	11.56	0.455	-2.54	-0.100
1L-17	4	27.75	3119	27.22	3060	0.358	0.026	9.75	0.384	12.78	0.503	-3.02	-0.119
1L-18	1	29.38	3302.5	28.85	3243.5	0.432	0.027	11.66	0.459	14.81	0.583	-3.15	-0.124
1L-18	2	29.38	3302.5	28.85	3243.5	0.454	0.028	12.24	0.482	15.37	0.605	-3.12	-0.123
1L-18	4	29.38	3302.5	28.85	3243.5	0.483	0.028	12.98	0.511	16.21	0.638	-3.23	-0.127
1L-19	1	31.01	3486	30.48	3427	0.561	0.028	14.96	0.589	18.47	0.727	-3.51	-0.138
1L-19	2	31.01	3486	30.48	3427	0.581	0.028	15.47	0.609	19.28	0.759	-3.81	-0.150
1L-19	4	31.01	3486	30.48	3427	0.622	0.029	16.54	0.651	20.37	0.802	-3.84	-0.151
1L-20	1	32.64	3669.5	32.11	3610.5	0.746	0.030	19.71	0.776	23.85	0.939	-4.14	-0.163
1L-20	2	32.64	3669.5	32.11	3610.5	0.797	0.030	21.01	0.827	25.48	1.003	-4.47	-0.176
1L-20	4	32.64	3669.5	32.11	3610.5	0.874	0.030	22.96	0.904	27.43	1.08	-4.47	-0.176
1L-21	1	33.76	3795	33.23	3736	1.098	0.030	28.65	1.128	33.66	1.325	-5.00	-0.197
1L-21	2	33.87	3807	33.34	3748	1.191	0.030	31.01	1.221	36.07	1.42	-5.05	-0.199
1L-21	4	34.30	3856	33.77	3797	1.380	0.029	35.79	1.409	41.48	1.633	-5.69	-0.224
1L-21	7	33.76	3795	33.23	3736	1.683	0.028	43.46	1.711	48.82	1.922	-5.36	-0.211
1U-1	1	20.67	2324	20.14	2265	1.712	0.019	43.97	1.731	50.88	2.003	-6.91	-0.272
1U-1	2	20.67	2324	20.14	2265	1.711	0.019	43.94	1.730	50.77	1.999	-6.83	-0.269
1U-1	4	20.67	2324	20.14	2265	1.708	0.019	43.87	1.727	50.42	1.985	-6.55	-0.258
1U-2	1	15.23	1712.5	14.70	1653.5	1.642	0.014	42.06	1.656	48.06	1.892	-5.99	-0.236
1U-2	2	15.23	1712.5	14.70	1653.5	1.638	0.014	41.96	1.652	47.90	1.886	-5.94	-0.234
1U-2	4	15.23	1712.5	14.70	1653.5	1.634	0.014	41.86	1.648	47.88	1.885	-6.02	-0.237
1U-3	1	9.79	1101	9.26	1042	1.533	0.010	39.19	1.543	44.50	1.752	-5.31	-0.209
1U-3	2	9.79	1101	9.26	1042	1.519	0.010	38.84	1.529	43.99	1.732	-5.16	-0.203
1U-3	4	9.79	1101	9.26	1042	1.513	0.009	38.66	1.522	43.87	1.727	-5.21	-0.205
1U-4	1	4.35	489.5	3.82	430.5	1.313	0.005	33.48	1.318	37.52	1.477	-4.04	-0.159
1U-4	2	4.35	489.5	3.82	430.5	1.303	0.005	33.22	1.308	37.41	1.473	-4.19	-0.165
1U-4	4	4.35	489.5	3.82	430.5	1.298	0.006	33.12	1.304	37.11	1.461	-3.99	-0.157
1U-5	1	0.00	0	-0.53	-59	0.992	0.001	25.22	0.993	28.47	1.121	-3.25	-0.128
1U-5	2	0.00	0	-0.53	-59	0.979	0.001	24.89	0.980	28.27	1.113	-3.38	-0.133
1U-5	4	0.00	0	-0.53	-59	0.964	0.000	24.49	0.964	27.71	1.091	-3.23	-0.127

Table B.3-Creep Data, Grandview, Missouri Test Site (English version).

	O-cell Load	Upward Mvment Top of Cell	Upward Mvment Top of Cell	Creep 2 to 4 min	Downward Mvment Bott of Cell	Downward Mvment Bott of Cell	Creep 2 to 4 min
	Net (Tons)	2 min (in)	4 min (in)	(in)	2 min (in)	4 min (in)	(in)
1L-0	0.0	0.000	0.000	0.000	0.000	0.000	0.000
1L-1	134.6	0.006	0.006	0.000	0.007	0.007	0.000
1L-2	308.0	0.007	0.007	0.000	0.007	0.007	0.000
1L-3	491.5	0.010	0.010	0.000	0.007	0.007	0.000
1L-4	675.0	0.012	0.012	0.000	0.006	0.007	0.001
1L-5	858.5	0.016	0.018	0.002	0.005	0.004	-0.001
1L-6	1042.0	0.021	0.022	0.001	0.004	0.004	0.000
1L-7	1225.5	0.027	0.029	0.002	0.015	0.013	-0.002
1L-8	1409.0	0.035	0.036	0.001	0.012	0.013	0.001
1L-9	1592.5	0.048	0.048	0.000	0.018	0.020	0.002
1L-10	1776.0	0.059	0.063	0.004	0.031	0.029	-0.002
1L-11	1959.5	0.081	0.086	0.005	0.040	0.038	-0.002
1L-12	2143.0	0.107	0.113	0.006	0.043	0.042	-0.001
1L-13	2326.6	0.140	0.146	0.006	0.046	0.061	0.015
1L-14	2509.5	0.183	0.192	0.009	0.064	0.061	-0.003
1L-15	2693.0	0.230	0.243	0.013	0.080	0.075	-0.005
1L-16	2876.5	0.292	0.310	0.018	0.088	0.098	0.010
1L-17	3060.0	0.355	0.384	0.029	0.100	0.119	0.019
1L-18	3243.5	0.482	0.511	0.029	0.123	0.127	0.004
1L-19	3427.0	0.609	0.651	0.042	0.150	0.151	0.001
1L-20	3610.5	0.827	0.904	0.077	0.176	0.176	0.000
1L-21	3797.0	1.221	1.409	0.188	0.199	0.224	0.025

Table B.4-Creep Data, Grandview, Missouri Test Site (Metric version).

	O-cell Load	Upward Mvment Top of Cell	Upward Mvment Top of Cell	Creep 2 to 4 min	Downward Mvment Bott of Cell	Downward Mvment Bott of Cell	Creep 2 to 4 min
	Net (MN)	2 min (mm)	4 min (mm)	(mm)	2 min (mm)	4 min (mm)	(mm)
1L-0	0.0	0.000	0.000	0.000	0.000	0.000	0.000
1L-1	1.2	0.152	0.152	0.000	0.178	0.178	0.000
1L-2	2.7	0.178	0.178	0.000	0.178	0.178	0.000
1L-3	4.4	0.254	0.254	0.000	0.178	0.178	0.000
1L-4	6.0	0.305	0.305	0.000	0.152	0.178	0.025
1L-5	7.6	0.406	0.457	0.051	0.127	0.102	-0.025
1L-6	9.3	0.533	0.559	0.025	0.102	0.102	0.000
1L-7	10.9	0.686	0.737	0.051	0.381	0.330	-0.051
1L-8	12.5	0.889	0.914	0.025	0.305	0.330	0.025
1L-9	14.2	1.219	1.219	0.000	0.457	0.508	0.051
1L-10	15.8	1.499	1.600	0.102	0.787	0.737	-0.051
1L-11	17.4	2.057	2.184	0.127	1.016	0.965	-0.051
1L-12	19.1	2.718	2.870	0.152	1.092	1.067	-0.025
1L-13	20.7	3.556	3.708	0.152	1.168	1.549	0.381
1L-14	22.3	4.648	4.877	0.229	1.626	1.549	-0.076
1L-15	24.0	5.842	6.172	0.330	2.032	1.905	-0.127
1L-16	25.6	7.417	7.874	0.457	2.235	2.489	0.254
1L-17	27.2	9.017	9.754	0.737	2.540	3.023	0.483
1L-18	28.9	12.243	12.979	0.737	3.124	3.226	0.102
1L-19	30.5	15.469	16.535	1.067	3.810	3.835	0.025
1L-20	32.1	21.006	22.962	1.956	4.470	4.470	0.000
1L-21	33.8	31.013	35.789	4.775	5.055	5.690	0.635

Table B.5- Strain gage data, Grandview, Missouri test site (English Units).

Load Test Incre.	Top of Conc Avg. Net Load (tons)	Level 6 Avg. Net Load (tons)	Level 5 Avg. Net Load (tons)	Level 4 Avg. Net Load (tons)	O-Cell^{TM} Gross Load (tons)	Level 1 Avg. Gross Load (tons)	Tip Avg. Load (tons)
EL (ft)	934	927	921	916.5	905	898	893.4
1L-0	0	0	0	0	0	0	0
1L-1	0	20.5	35.5	60	183.5	35	0
1L-3	0	40.0	72	126	550.5	73.5	0
1L-5	0	67	122	211	917.5	116.5	0
1L-7	0	110.5	202.5	338	1284.5	170.5	0
1L-9	0	167.5	312.5	510.5	1651.5	243	0
1L-11	0	227	440.5	722	2153.5	324	0
1L-13	0	274.5	563.5	940	2385.5	433	0
1L-15	0	310	683	1131	2752	514.5	0
1L-17	0	348.5	789.5	1263	3119	550	0
1L-19	0	407.5	890	1353.5	3486	501	0
1L-21	0	446.5	805.5	1230	3856	498	0
Level 2 & 3 strain gages yielded unusual/ unreliable data and are not included							

Table B.6- Strain gage data, Grandview, Missouri test site (Metric Units).

Load Test Incre.	Top of Conc Avg. Net Load (MN)	Level 6 Avg. Net Load (MN)	Level 5 Avg. Net Load (MN)	Level 4 Avg. Net Load (MN)	O-Cell^{TM} Gross Load (MN)	Level 1 Avg. Gross Load (MN)	Tip Avg. Load (MN)
EL (m)	284.7	282.5	280.7	279.3	275.8	273.7	272.3
1L-0	0	0.00	0.00	0	0	0	0
1L-1	0	0.18	0.32	0.53	1.63	0.31	0
1L-3	0	0.36	0.64	1.12	4.90	0.65	0
1L-5	0	0.60	1.09	1.88	8.16	1.04	0
1L-7	0	0.98	1.80	3.01	11.40	1.52	0
1L-9	0	1.49	2.78	4.54	14.69	2.16	0
1L-11	0	2.02	3.92	6.42	19.16	2.88	0
1L-13	0	2.44	5.01	8.36	21.22	3.85	0
1L-15	0	2.76	6.08	10.06	24.48	4.58	0
1L-17	0	3.10	7.02	11.24	27.75	4.89	0
1L-19	0	3.63	7.92	12.04	31.01	4.46	0
1L-21	0	3.97	7.17	10.94	34.30	4.43	0
Level 2 & 3 strain gages yielded unusual/ unreliable data and are not included							

Table B.7- Unit strain gage data, Grandview, Missouri test site (English Units).

Load	O-cell Loads		Upwrd	Bottom	Tip to SG-1	Level 1	SG-1 to O-cell	Level 4
				of	Wea	Avg.	Westerville	Avg.
Test	Gross	Net	Mvment	Cell	893.4 to 898	Load	898 to 905	Load
Incre.	(tons)	(tons)	(in)	(in)	(tsf)	(tons)	(tsf)	(tons)
1L-0	0	0	0	0	0	0	0	0
1L-1	184	125	0.006	0.007	0.2	35	1.1	60
1L-2	367	308	0.007	0.007	0.6	54.5	2.2	90.5
1L-3	551	492	0.010	0.007	0.9	73.5	3.4	126
1L-4	734	675	0.012	0.007	1.1	90.5	4.6	157
1L-5	918	859	0.018	0.004	1.4	116.5	5.7	211
1L-6	1101	1042	0.021	0.004	2.2	190	6.5	266.5
1L-7	1285	1226	0.028	0.014	2.0	170.5	8.0	338
1L-8	1468	1409	0.035	0.013	2.4	205	9.0	421.5
1L-9	1652	1593	0.048	0.019	2.8	243	10.1	510.5
1L-10	1835	1776	0.063	0.030	3.3	283	11.1	606
1L-11	2019	1960	0.086	0.038	3.8	324	12.1	722
1L-12	2202	2143	0.114	0.041	4.4	377	13.1	830.5
1L-13	2386	2327	0.147	0.060	5.0	433	14.0	940
1L-14	2569	2510	0.192	0.061	5.6	483	14.9	1049.5
1L-15	2752	2693	0.243	0.075	6.0	514.5	16.0	1131
1L-16	2936	2877	0.309	0.098	6.3	540	17.1	1205
1L-17	3119	3060	0.384	0.119	6.4	550	18.4	1263
1L-18	3303	3244	0.511	0.127	6.1	523.5	19.9	1313
1L-19	3486	3427	0.650	0.152	5.8	501	21.4	1353.5
1L-20	3670	3611	0.904	0.177	5.7	491	22.7	1382.5
1L-21	3856	3797	1.409	0.211	5.9	508	24.0	1299.5

Table B.8- Unit strain gage data, Grandview, Missouri test site (English Units)

Load Test Incre.	O-cell to SG-4 Quiv&Cem C 905 to 916.5 (tsf)	Level 5 Avg. Load (tons)	SG-4 to SG-5 Chanute 916.5 to 921 (tsf)	Level 6 Avg. Load (tons)	SG-5 to SG-6 Chanute 921 to 927 (tsf)	SG-6 to Top Chanute 927 to 934 (tsf)	SG-4 to Top Chanute 916.5 to 934 (tsf)
1L-0	0	0	0	0	0	0	0
1L-1	0.5	36	0.2	21	0.0	0.0	0.1
1L-2	1.1	52	0.3	30	0.1	0.1	0.2
1L-3	1.7	72	0.5	40	0.1	0.2	0.3
1L-4	2.4	90	0.6	50	0.2	0.2	0.4
1L-5	2.9	122	0.9	67	0.3	0.3	0.6
1L-6	3.5	157	1.1	86	0.5	0.5	0.7
1L-7	4.0	203	1.4	111	0.6	0.7	0.9
1L-8	4.4	255	1.7	140	0.8	0.9	1.2
1L-9	4.8	313	2.1	168	1.1	1.1	1.4
1L-10	5.2	373	2.4	196	1.3	1.3	1.7
1L-11	5.5	441	3.0	227	1.6	1.5	2.0
1L-12	5.8	504	3.5	254	1.9	1.7	2.3
1L-13	6.1	564	4.0	275	2.2	1.8	2.6
1L-14	6.4	626	4.5	295	2.6	1.9	2.9
1L-15	6.9	683	4.8	310	2.9	2.1	3.1
1L-16	7.3	739	5.0	327	3.2	2.2	3.4
1L-17	7.9	790	5.1	349	3.5	2.3	3.5
1L-18	8.4	845	5.0	375	3.7	2.5	3.7
1L-19	9.0	890	5.0	408	3.8	2.7	3.8
1L-20	9.7	917	5.0	443	3.8	3.0	3.9
1L-21	10.9	855	4.8	463	3.1	3.1	3.6

Table B.9- Unit strain gage data, Grandview, Missouri test site (Metric Units).

Load	O-cell Loads		Upwrd	Bottom	Tip to SG-1	Level 1	SG-1 to O-cell	Level 4
				of	Wea	Avg.	Westerville	Avg.
Test	Gross	Net	Mvment	Cell	272.3-273.7	Load	273.7-275.8	Load
Incre.	(MN)	(MN)	(mm)	(mm)	(kPa)	(MN)	(kPa)	(MN)
1L-0	0	0	0	0	0	0.00	0	0
1L-1	1.63	1.11	0.2	0.2	14.5	0.31	101.8	0.53
1L-2	3.26	2.74	0.2	0.2	60.8	0.48	214.1	0.81
1L-3	4.90	4.37	0.3	0.2	82.0	0.65	326.9	1.12
1L-4	6.53	6.00	0.3	0.2	101.0	0.81	440.9	1.40
1L-5	8.16	7.64	0.5	0.1	130.0	1.04	548.9	1.88
1L-6	9.79	9.27	0.5	0.1	211.9	1.69	624.2	2.37
1L-7	11.43	10.90	0.7	0.4	190.2	1.52	763.3	3.01
1L-8	13.06	12.53	0.9	0.3	228.7	1.82	865.4	3.75
1L-9	14.69	14.17	1.2	0.5	271.1	2.16	965.1	4.54
1L-10	16.32	15.80	1.6	0.8	315.7	2.52	1063.5	5.39
1L-11	17.96	17.43	2.2	1.0	361.4	2.88	1161.1	6.42
1L-12	19.59	19.06	2.9	1.0	420.5	3.35	1250.5	7.39
1L-13	21.22	20.70	3.7	1.5	483.0	3.85	1337.9	8.36
1L-14	22.85	22.32	4.9	1.5	538.8	4.30	1429.0	9.34
1L-15	24.48	23.96	6.2	1.9	573.9	4.58	1533.2	10.06
1L-16	26.11	25.59	7.8	2.5	602.4	4.80	1641.5	10.72
1L-17	27.75	27.22	9.8	3.0	613.5	4.89	1760.4	11.24
1L-18	29.38	28.85	13.0	3.2	584.0	4.66	1904.3	11.68
1L-19	31.01	30.49	16.5	3.9	558.9	4.46	2045.4	12.04
1L-20	32.64	32.12	23.0	4.5	547.7	4.37	2178.0	12.30
1L-21	34.30	33.78	35.8	5.4	566.7	4.52	2294.1	11.56

For upward loaded shear, the bouyant weight of the shaft in each zone has been subtracted from the load shed in the respective zone above the O-cell^{TM}

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Table B.10- Unit strain gage data, Grandview, Missouri test site (Metric Units).

Load Test Incre.	O-cell to SG-4 Quiv&Cem C 275.8-279.3 (kPa)	Level 5 Avg. Load (MN)	SG-4 to SG-5 Chanute 279.3-280.7 (kPa)	Level 6 Avg. Load (MN)	SG-5 to SG-6 Chanute 280.7-282.5 (kPa)	SG-6 to Top Chanute 282.5-284.7 (kPa)	SG-4 to Top Chanute 282.5-284.7 (kPa)
1L-0	0	0.00	0	0.00	0	0	0
1L-1	43.7	0.32	13.7	0.18	0.1	2.1	13.5
1L-2	106.4	0.46	27.3	0.26	5.6	8.2	21.7
1L-3	166.9	0.64	41.4	0.36	13.5	15.2	31.2
1L-4	229.3	0.80	53.5	0.44	20.1	21.6	39.6
1L-5	282.3	1.09	74.1	0.60	31.5	33.4	54.1
1L-6	334.7	1.39	93.8	0.77	43.7	46.2	69.0
1L-7	380.5	1.80	117.7	0.98	60.5	62.6	88.3
1L-8	421.5	2.27	146.7	1.25	78.6	82.5	110.7
1L-9	460.1	2.78	176.2	1.49	102.1	101.0	134.6
1L-10	496.2	3.31	209.5	1.74	127.2	119.8	160.3
1L-11	523.8	3.92	254.4	2.02	155.8	141.0	191.5
1L-12	554.5	4.48	296.6	2.26	184.8	158.8	220.7
1L-13	584.8	5.01	343.4	2.44	215.0	172.9	250.2
1L-14	614.8	5.57	387.4	2.62	248.4	186.4	279.6
1L-15	656.6	6.08	410.4	2.76	280.9	196.8	301.5
1L-16	701.4	6.57	427.7	2.91	311.1	208.2	321.4
1L-17	752.8	7.02	434.3	3.10	334.3	222.7	337.0
1L-18	807.4	7.51	429.6	3.33	357.0	240.1	350.5
1L-19	865.9	7.92	424.9	3.63	366.8	262.3	361.4
1L-20	929.1	8.15	427.3	3.94	359.8	286.2	369.2
1L-21	1039.4	7.60	407.6	4.12	295.4	299.6	346.8

Table B.11- Unconfined compressive strength of NX rock cores

	Upper Chanute	Lower Chanute	Cement City	Quivira	Westerville	Wea
	(tsf)	(tsf)	(tsf)	(tsf)	(tsf)	(tsf)
	12.4	5.2	240.7	11.1	861.0	32.6
	8.5	4.6	571.4	14.3	725.8	19.4
	8.5	2.7	118.6	2.9*	164.8	13.8
		9.7	458.3	16.0	411.6	32.7
		10.1	410.2	18.2	884.2	18.9
		11.0	319.0		1104.4	27.6
			588.0		137.9	20.7
					707.3	25.9
					786.2	20.0
					800.9	33.1
					500.7	8.4
					800.9	29.9
						25.6
						24.9
						30.5
						17.5
						32.1
						20.1
						1.4*
Mean	9.8	7.2	386.6	14.9	657.1	24.1
Std. Dev.	2.3	3.5	179.8	3.0	311.9	8.7

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Table B.12- Unconfined compressive strength and SPT data for Chanute Formation, Cement City , and Quivira.

Pier	Elev.	Unconfined Compressive Strength and SPT Data			
		A6251	A6252	Test Shaft	A6254
	(ft)	(tsf)	(tsf)	(tsf)	(tsf)
		Chanute			
TS	937.1			100 in 10.5"	
11	936.8				12.4
TS	932.1			100 in 7"	8.5
11	930.2				8.5
7	927.6	9.7			
TS	927.1			100 in 3"	
11	923.9				11
13	921.3	2.7			
TS	920.0			5.2	
7	919.5		10.1		
TS	918.3			4.6	
		Cement City			
8	916.1		571.4		
13	915.2	458.3			
5	915.0		240.7		
TS	914.8			118.6	
TS	913.1			319.0	
4	912.0		410.2		
TS	910.9			588.0	
		Quivira			
11	913.1				18.2
8	910.1		11.1		
TS	910.0			14.3	
6	906.9		16.0		
TS	906.1			2.9*	

* Values not used in calculation of mean

Table B.13- Unconfined compressive strength and SPT data for Westerville Limestone and Wea Shale Formation.

Pier	Elev.	Unconfined Compressive Strength and SPT Data			
		A6251	A6252	Test Shaft	A6254
	(ft)	(tsf)	(tsf)	(tsf)	(tsf)
		Westerville			
9	908.3				500.7
9	905.0				800.9
4	904.9		137.9		
8	904.1				1104.4
11	903.5	786.2			
TS	903.1			164.8	
8	901.1		725.8		
12	900.9	884.2			
4	900.7		707.3		
5	899.8		861.0		
TS	899.0			411.6	
		Wea			
8	896.7				20.1
5	896.6		17.5		
4	896.5		8.4		
TS	896.0			13.8	
6	896.0		20		
8	892.9		19.4		
TS	892.7			32.7	
6	891.4		33.1		
5	891.0		32.6		
TS	890.8			13.9	
8	890.1				32.1
TS	887.7			27.6	
4	886.6		29.9		
4	882.6		25.6		
TS	881.0			20.7	
10	877.4	50 in 6"			
4	876.4		24.9		
TS	874.3			25.9	
10	874.2	1.37*			
10	873.6		100 in 12"		
4	872.5		30.5		

* Values not used in calculation of mean

APPENDIX C
DETAILED DATA FOR LOAD TESTS AND UNCONFINED COMPRESSIVE
STRENGTH OF NX CORES AT WAVERLY SITE

Table C.1
Summary of Dimensions, Elevations, and Shaft Properties

Shaft:

Average Shaft Diameter (EL 937.5 ft to 607.6 ft)	=	2134 mm	84 in
Average Shaft Diameter (EL 607.6 ft to 558.0 ft)	=	1981 mm	78 in
O-cell^{TM}: 1004-18A	=	660 mm	26 in
Length of side shear above break at base of O-cell^{TM}	=	22.52 m	73.9 ft
Length of side shear below break at base of O-cell^{TM}	=	1.7 m	5.6 ft
Shaft side shear area above O-cell^{TM} base	=	144.5 m^2	1556.0 ft^2
Shaft side shear area below break at base of O-cell^{TM}	=	10.59 m^2	113.97 ft^2
Shaft base area	=	3.08 m^2	33.2 ft^2
Bouyant weight of shaft above base of O-cell^{TM}	=	1.03 MN	230.6 kips
Estimated shaft stiffness (EL 637.5 ft to 607.6 ft) =		141.5 GN	31,800,000 kips
Estimated shaft stiffness (EL 607.6 ft to 558 ft) =		110.7 GN	24,900,000 kips
Elevation of Water Table	=	200.4 m	657.5 ft
Elevation of Mud line	=	194.3 m	637.5 ft
Elevation top of shaft concrete	=	191.3 m	627.5 ft
Elevation of base of O-cell^{TM}	=	171.8 m	563.6 ft
Elevation of shaft tip	=	170.1 m	558.0 ft

Casing:

Elevation of top of temporary casing (2134 mm O.D.)	=	202.8 m	665.5 ft
Elevation of bottom of temporary casing (84 in O.D.)	=	185.2 m	607.6 ft

Compression Sections:

EL. of top of telltale used for upper shaft compression	=	191.9 m	629.5 ft
EL. of bottom of telltale used upper shaft compression	=	172.2 m	565.0 ft

Strain Gages:

Elevation of strain gage level 4	=	182.8 m	599.6 ft
Elevation of strain gage level 3	=	178.2 m	584.6 ft
Elevation of strain gage level 2	=	175.1 m	574.6 ft
Elevation of strain gage level 1	=	170.9 m	560.6 ft

Miscellaneous:

Top Plate Diameter	=	1537 mm	60.5 in
Bottom Plate Diameter	=	1537 mm	60.5 in
Vertical Rebar size	=	M 45	#14
Number of vertical bars	=	22	
Hoop re-bar size	=	M 16	# 5
Unconfined compressive concrete strength	=	51.8 MPa	7529 psi

Table C.2- Osterberg O-cell^{TM} versus top and bottom plate movement for load increments 1L-0 to 1U-4.

Load	Hold	O-cell Loads					Top of	Upper	Upward Movement		O-cell expansion		Dnwrd Mvment	
Test	Time	Gross	Gross	Gross	Net	Net	Shaft	Compre	Top Plate					
Incre	Min	(MN)	(kips)	(tons)	(MN)	(tons)	(in)	(in)	(mm)	(in)	(mm)	(in)	(mm)	(in)
							A	B	A + B		C		A + B - C	
1L-0		0.00	0	0	0	0	0	0	0.00	0	0	0	0	0
1L-1	1	1.06	239	119.5	0.03	4.2	0.002	0.001	0.08	0.003	0.15	0.006	-0.08	-0.003
1L-1	2	1.06	239	119.5	0.03	4.2	0.001	0.001	0.05	0.002	0.15	0.006	-0.10	-0.004
1L-1	4	1.06	239	119.5	0.03	4.2	0.001	0.001	0.05	0.002	0.15	0.006	-0.10	-0.004
1L-2	1	2.04	458	229	1.01	113.7	0.002	0.001	0.08	0.003	0.20	0.008	-0.13	-0.005
1L-2	2	2.04	458	229	1.01	113.7	0.002	0.001	0.08	0.003	0.20	0.008	-0.13	-0.005
IL-2	4	2.04	458	229	1.01	113.7	0.002	0.001	0.08	0.003	0.20	0.008	-0.13	-0.005
1L-3	1	3.01	677	338.5	1.98	223.2	0.003	0.002	0.13	0.005	0.25	0.01	-0.13	-0.005
1L-3	2	3.01	677	338.5	1.98	223.2	0.003	0.001	0.10	0.004	0.25	0.01	-0.15	-0.006
1L-3	4	3.01	677	338.5	1.98	223.2	0.003	0.002	0.13	0.005	0.25	0.01	-0.13	-0.005
1L-4	1	3.98	895	447.5	2.95	332.2	0.003	0.002	0.13	0.005	0.33	0.013	-0.20	-0.008
1L-4	2	3.98	895	447.5	2.95	332.2	0.003	0.002	0.13	0.005	0.33	0.013	-0.20	-0.008
1L-4	4	3.98	895	447.5	2.95	332.2	0.003	0.002	0.13	0.005	0.36	0.014	-0.23	-0.009
1L-5	1	4.96	1114	557	3.93	441.7	0.004	0.003	0.18	0.007	0.43	0.017	-0.25	-0.010
1L-5	2	4.96	1114	557	3.93	441.7	0.004	0.002	0.15	0.006	0.46	0.018	-0.30	-0.012
1L-5	4	4.96	1114	557	3.93	441.7	0.004	0.002	0.15	0.006	0.46	0.018	-0.30	-0.012
1L-6	1	5.93	1333	666.5	4.90	551.2	0.004	0.003	0.18	0.007	0.56	0.022	-0.38	-0.015
1L-6	2	5.93	1333	666.5	4.90	551.2	0.004	0.003	0.18	0.007	0.56	0.022	-0.38	-0.015
1L-6	4	5.93	1333	666.5	4.90	551.2	0.005	0.003	0.20	0.008	0.58	0.023	-0.38	-0.015
1L-7	1	6.90	1551	775.5	5.87	660.2	0.005	0.004	0.23	0.009	0.66	0.026	-0.43	-0.017
1L-7	2	6.90	1551	775.5	5.87	660.2	0.006	0.004	0.25	0.010	0.69	0.027	-0.43	-0.017
1L-7	4	6.90	1551	775.5	5.87	660.2	0.005	0.004	0.23	0.009	0.71	0.028	-0.48	-0.019
1L-8	1	7.87	1770	885	6.84	769.7	0.005	0.004	0.23	0.009	0.79	0.031	-0.56	-0.022
1L-8	2	7.87	1770	885	6.84	769.7	0.006	0.005	0.28	0.011	0.81	0.032	-0.53	-0.021
1L-8	4	7.87	1770	885	6.84	769.7	0.006	0.005	0.28	0.011	0.84	0.033	-0.56	-0.022
1L-9	1	8.84	1988	994	7.81	878.7	0.007	0.005	0.30	0.012	0.91	0.036	-0.61	-0.024
1L-9	2	8.84	1988	994	7.81	878.7	0.007	0.006	0.33	0.013	0.94	0.037	-0.61	-0.024
1L-9	4	8.84	1988	994	7.81	878.7	0.007	0.006	0.33	0.013	0.94	0.037	-0.61	-0.024
1L-10	1	9.82	2207	1103.5	8.79	988.2	0.008	0.006	0.36	0.014	1.04	0.041	-0.69	-0.027
1L-10	2	9.82	2207	1103.5	8.79	988.2	0.008	0.007	0.38	0.015	1.07	0.042	-0.69	-0.027
1L-10	4	9.82	2207	1103.5	8.79	988.2	0.008	0.007	0.38	0.015	1.07	0.042	-0.69	-0.027
1L-11	1	10.79	2426	1213	9.76	1097.7	0.009	0.008	0.43	0.017	1.17	0.046	-0.74	-0.029
1L-11	2	10.79	2426	1213	9.76	1097.7	0.009	0.008	0.43	0.017	1.19	0.047	-0.76	-0.030
1L-11	4	10.79	2426	1213	9.76	1097.7	0.009	0.008	0.43	0.017	1.22	0.048	-0.79	-0.031
1L-12	1	11.76	2644	1322	10.73	1206.7	0.009	0.008	0.43	0.017	1.30	0.051	-0.86	-0.034
1L-12	2	11.76	2644	1322	10.73	1206.7	0.009	0.009	0.46	0.018	1.32	0.052	-0.86	-0.034
1L-12	4	11.76	2644	1322	10.73	1206.7	0.010	0.009	0.48	0.019	1.32	0.052	-0.84	-0.033
1L-13	1	12.73	2863	1431.5	11.70	1316.2	0.010	0.010	0.51	0.020	1.42	0.056	-0.91	-0.036
1L-13	2	12.73	2863	1431.5	11.70	1316.2	0.010	0.010	0.51	0.020	1.45	0.057	-0.94	-0.037
1L-13	4	12.73	2863	1431.5	11.70	1316.2	0.011	0.010	0.53	0.021	1.45	0.057	-0.91	-0.036

Table C.2- Continued.

Load	Hold	O-cell Loads					Top of	Upper	Upward Movement		O-cell expansion		Dnwrd Mvment	
Test	Time	Gross	Gross	Gross	Net	Net	Shaft	Compre	Top Plate					
Incre	Min	(MN)	(kips)	(tons)	(MN)	(tons)	(in)	(in)	(mm)	(in)	(mm)	(in)	(mm)	(in)
							A	B	A + B		C		A + B - C	
1L-14	1	13.71	3082	1541	12.68	1425.7	0.011	0.011	0.56	0.022	1.55	0.061	-0.99	-0.039
1L-14	2	13.71	3082	1541	12.68	1425.7	0.010	0.011	0.53	0.021	1.57	0.062	-1.04	-0.041
1L-14	4	13.71	3082	1541	12.68	1425.7	0.011	0.011	0.56	0.022	1.60	0.063	-1.04	-0.041
1L-15	1	14.68	3300	1650	13.65	1534.7	0.012	0.012	0.61	0.024	1.70	0.067	-1.09	-0.043
1L-15	2	14.68	3300	1650	13.65	1534.7	0.012	0.012	0.61	0.024	1.70	0.067	-1.09	-0.043
1L-15	4	14.68	3300	1650	13.65	1534.7	0.012	0.012	0.61	0.024	1.73	0.068	-1.12	-0.044
1L-16	1	15.65	3519	1759.5	14.62	1644.2	0.013	0.014	0.69	0.027	1.83	0.072	-1.14	-0.045
1L-16	2	15.65	3519	1759.5	14.62	1644.2	0.013	0.014	0.69	0.027	1.85	0.073	-1.17	-0.046
1L-16	4	15.65	3519	1759.5	14.62	1644.2	0.013	0.014	0.69	0.027	1.88	0.074	-1.19	-0.047
1L-17	1	16.63	3738	1869	15.60	1753.7	0.014	0.015	0.74	0.029	1.98	0.078	-1.24	-0.049
1L-17	2	16.63	3738	1869	15.60	1753.7	0.015	0.015	0.76	0.030	2.01	0.079	-1.24	-0.049
1L-17	4	16.63	3738	1869	15.60	1753.7	0.014	0.015	0.74	0.029	2.03	0.08	-1.30	-0.051
1L-18	1	17.60	3956	1978	16.57	1862.7	0.015	0.016	0.79	0.031	2.13	0.084	-1.35	-0.053
1L-18	2	17.60	3956	1978	16.57	1862.7	0.016	0.016	0.81	0.032	2.18	0.086	-1.37	-0.054
1L-18	4	17.60	3956	1978	16.57	1862.7	0.015	0.017	0.81	0.032	2.21	0.087	-1.40	-0.055
1L-19	1	18.57	4175	2087.5	17.54	1972.2	0.016	0.017	0.84	0.033	2.31	0.091	-1.47	-0.058
1L-19	2	18.57	4175	2087.5	17.54	1972.2	0.017	0.017	0.86	0.034	2.34	0.092	-1.47	-0.058
1L-19	4	18.57	4175	2087.5	17.54	1972.2	0.017	0.018	0.89	0.035	2.46	0.097	-1.57	-0.062
1L-20	1	19.54	4394	2197	18.51	2081.7	0.018	0.018	0.91	0.036	2.51	0.099	-1.60	-0.063
1L-20	2	19.54	4394	2197	18.51	2081.7	0.019	0.018	0.94	0.037	2.51	0.099	-1.57	-0.062
1L-20	4	19.54	4394	2197	18.51	2081.7	0.019	0.018	0.94	0.037	2.54	0.1	-1.60	-0.063
1L-21	1	20.51	4612	2306	19.48	2190.7	0.019	0.019	0.97	0.038	2.67	0.105	-1.70	-0.067
1L-21	2	20.51	4612	2306	19.48	2190.7	0.020	0.019	0.99	0.039	2.69	0.106	-1.70	-0.067
1L-21	4	20.51	4612	2306	19.48	2190.7	0.021	0.019	1.02	0.040	2.72	0.107	-1.70	-0.067
1L-22	1	21.49	4831	2415.5	20.46	2300.2	0.021	0.020	1.04	0.041	2.87	0.113	-1.83	-0.072
1L-22	2	21.49	4831	2415.5	20.46	2300.2	0.022	0.020	1.07	0.042	2.90	0.114	-1.83	-0.072
1L-22	4	21.49	4831	2415.5	20.46	2300.2	0.022	0.020	1.07	0.042	2.92	0.115	-1.85	-0.073
1L-23	1	22.46	5049	2524.5	21.43	2409.2	0.023	0.021	1.12	0.044	3.02	0.119	-1.91	-0.075
1L-23	2	22.46	5049	2524.5	21.43	2409.2	0.023	0.021	1.12	0.044	3.05	0.12	-1.93	-0.076
1L-23	3	22.46	5049	2524.5	21.43	2409.2	0.023	0.021	1.12	0.044	3.05	0.12	-1.93	-0.076
1L-23	4	22.46	5049	2524.5	21.43	2409.2	0.023	0.021	1.12	0.044	3.10	0.122	-1.98	-0.078
1U-1	1	14.68	3300	1650	13.65	1534.7	0.021	0.019	1.02	0.040	2.59	0.102	-1.57	-0.062
1U-1	2	14.68	3300	1650	13.65	1534.7	0.021	0.019	1.02	0.040	2.57	0.101	-1.55	-0.061
1U-1	4	14.68	3300	1650	13.65	1534.7	0.022	0.019	1.04	0.041	2.57	0.101	-1.52	-0.060
1U-2	1	9.82	2207	1103.5	8.79	988.2	0.020	0.017	0.94	0.037	2.13	0.084	-1.19	-0.047
1U-2	2	9.82	2207	1103.5	8.79	988.2	0.019	0.017	0.91	0.036	2.11	0.083	-1.19	-0.047
1U-2	4	9.82	2207	1103.5	8.79	988.2	0.019	0.017	0.91	0.036	2.08	0.082	-1.17	-0.046
1U-3	2	4.96	1114	557	3.93	441.7	0.017	0.013	0.76	0.030	1.55	0.061	-0.79	-0.031
1U-3	4	4.96	1114	557	3.93	441.7	0.017	0.013	0.76	0.030	1.52	0.06	-0.76	-0.030
1U-4	2	0.00	0	0	-1.03	0	0.014	0.010	0.61	0.024	0.94	0.037	-0.33	-0.013

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| 1U-4 | 4 | 0.00 | 0 | 0 | -1.03 | 0 | 0.014 | 0.010 | 0.61 | 0.024 | 0.91 | 0.036 | -0.30 | -0.012 |

Table C.3-Creep data, Waverly, Missouri test site.

Load Incre.	O-cell Load	Upward Movement Top of Upper cell	Upward Movement Top of Upper cell	Creep 2 to 4 min	Downward Movement Bott of Upper cell	Downward Movement Bott of Upper cell	Creep 2 to 4 min
	Net (Tons)	2 min (in)	4 min (in)	(in)	2 min (in)	4 min (in)	(in)
1L-0	0.0	0.000	0.000	0.000	0.000	0.000	0.000
1L-1	119.5	0.003	0.003	0.000	0.004	0.004	0.000
1L-2	229.5	0.004	0.004	0.000	0.004	0.004	0.000
1L-3	338.5	0.005	0.005	0.000	0.005	0.006	0.001
1L-4	447.5	0.005	0.005	0.000	0.008	0.008	0.000
1L-5	556.5	0.007	0.007	0.000	0.011	0.012	0.001
1L-6	666.5	0.008	0.008	0.000	0.014	0.015	0.001
1L-7	775.5	0.010	0.010	0.000	0.017	0.018	0.001
1L-8	885.0	0.011	0.012	0.001	0.021	0.021	0.000
1L-9	994.0	0.013	0.014	0.001	0.024	0.024	0.000
1L-10	1103.5	0.015	0.016	0.001	0.026	0.026	0.000
1L-11	1213.0	0.017	0.018	0.001	0.029	0.030	0.001
1L-12	1322.0	0.019	0.020	0.001	0.033	0.033	0.000
1L-13	1431.5	0.021	0.021	0.000	0.036	0.036	0.000
1L-14	1541.0	0.022	0.023	0.001	0.040	0.040	0.000
1L-15	1650.0	0.025	0.025	0.000	0.043	0.043	0.000
1L-16	1759.5	0.027	0.027	0.000	0.046	0.047	0.001
1L-17	1869.0	0.029	0.030	0.001	0.049	0.050	0.001
1L-18	1978.0	0.032	0.032	0.000	0.054	0.055	0.001
1L-19	2087.5	0.034	0.035	0.001	0.058	0.062	0.004
1L-20	2197.0	0.036	0.037	0.001	0.063	0.063	0.000
1L-21	2306.0	0.039	0.039	0.000	0.067	0.068	0.001
1L-22	2415.5	0.041	0.042	0.001	0.072	0.072	0.000
1L-23	2524.5	0.044	0.043	-0.001	0.076	0.078	0.002

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Table C.4- Strain gage data, Waverly, Missouri test site (English Units).

Load Test Incre.	0 Shear Avg. Net Load (tons)	Level 4 Avg. Net Load (tons)	Level 3 Avg. Net Load (tons)	Level 2 Avg. Net Load (tons)	O-Cell™ Gross Load (tons)	Level 1 Avg. Gross Load (tons)
EL (ft)	606.5	599.6	584.6	574.6	563.6	560.6
1L-0	0	0	0	0	0	0
1L-1	0	7.5	29	65.5	119.5	51.5
1L-3	0	13.5	47	110	338.5	62.5
1L-5	0	22.5	77	192.5	557	88
1L-7	0	33	116.5	284.5	775.5	107.5
1L-9	0	43.5	154	374.5	994	120
1L-11	0	56.5	192	473	1213	133
1L-13	0	68	231	568.5	1431.5	117
1L-15	0	82	274.5	673.5	1650	150.5
1L-17	0	95.5	320	784	1869	135
1L-19	0	114.5	384.5	941	2087.5	131
1L-21	0	125.5	415.5	1015	2306	35
1I-23	0	140.5	462.5	1130.5	2524.5	-238.5

Table C.5- Strain gage data, Waverly, Missouri test site (Metric Units).

Load Test Incre.	Mud Line Avg. Net Load (MN)	Level 4 Avg. Net Load (MN)	Level 3 Avg. Net Load (MN)	Level 2 Avg. Net Load (MN)	O-Cell^{TM} Gross Load (MN)	Level 1 Avg. Gross Load (MN)
EL (m)	184.85	182.75	178.18	175.13	171.78	170.86
1L-0	0	0.00	0.00	0.00	0.0	0.0
1L-1	0	0.07	0.26	0.58	1.1	0.5
1L-3	0	0.12	0.42	0.98	3.0	0.6
1L-5	0	0.20	0.68	1.71	5.0	0.8
1L-7	0	0.29	1.04	2.53	6.9	1.0
1L-9	0	0.39	1.37	3.33	8.8	1.1
1L-11	0	0.50	1.71	4.21	10.8	1.2
1L-13	0	0.60	2.05	5.06	12.7	1.0
1L-15	0	0.73	2.44	5.99	14.7	1.3
1L-17	0	0.85	2.85	6.97	16.6	1.2
1L-19	0	1.02	3.42	8.37	18.6	1.2
1L-21	0	1.12	3.70	9.03	20.5	0.3
1L-23	0	1.25	4.11	10.06	22.5	-2.1

Table C.6- Unit strain gage data, Waverly, Missouri test site (English Units)

Load	O-cell Load		Upwrd	Bottom of	Level 2	O-cell to	Level 3	SG-2 to	Level 4	SG-3 to	SG-4 to
Test	Gross	Net	Mvment	Cell		SG-2		SG-3		SG-4	0 Shear
Incre.					Avg.	563.6 to	Avg.	574.6 to	Avg.	584.6 to	599.6 to
					Load	574.6	Load	584.6	Load	599.6	606.5
	(tons)	(tons)	(in)	(in)	(tons)	(tsf)	(tons)	(tsf)	(tons)	(tsf)	(tsf)
1L-0	0	0	0	0	0	0	0	0	0	0	0
1L-1	120	4	0.003	0.004	65.5	-0.3	29	0.1	8	0.01	-0.02
1L-2	229	114	0.004	0.004	85.5	0.1	38	0.2	10	0.02	-0.01
1L-3	339	223	0.005	0.006	110	0.4	47	0.2	14	0.04	0.02
1L-4	448	332	0.005	0.008	144	0.8	60	0.3	18	0.07	0.05
1L-5	557	442	0.007	0.012	192.5	1.0	80	0.5	23	0.12	0.08
1L-6	667	551	0.008	0.015	235.5	1.3	98	0.6	28	0.17	0.12
1L-7	776	660	0.010	0.018	284.5	1.6	117	0.8	33	0.21	0.16
1L-8	885	770	0.012	0.021	330	1.9	137	0.9	38	0.26	0.19
1L-9	994	879	0.014	0.024	374.5	2.2	154	1.0	44	0.30	0.23
1L-10	1104	988	0.016	0.026	424.5	2.4	173	1.2	49	0.34	0.27
1L-11	1213	1098	0.018	0.030	473	2.7	192	1.3	57	0.38	0.32
1L-12	1322	1207	0.020	0.033	519	3.0	211	1.4	61	0.42	0.35
1L-13	1432	1316	0.021	0.036	568.5	3.3	231	1.6	68	0.47	0.41
1L-14	1541	1426	0.023	0.040	619.5	3.5	252	1.7	73	0.52	0.44
1L-15	1650	1535	0.025	0.043	673.5	3.8	275	1.9	82	0.56	0.51
1L-16	1760	1644	0.027	0.047	730	4.0	298	2.1	88	0.62	0.55
1L-17	1869	1754	0.030	0.050	784	4.3	320	2.2	96	0.67	0.60
1L-18	1978	1863	0.032	0.055	843	4.5	344	2.4	103	0.72	0.65
1L-19	2088	1972	0.035	0.062	941	4.5	385	2.7	115	0.82	0.74
1L-20	2197	2082	0.037	0.063	959	4.9	393	2.7	118	0.83	0.76
1L-21	2306	2191	0.039	0.068	1015	5.2	416	2.9	126	0.88	0.81
1L-22	2416	2300	0.042	0.072	1070.5	5.4	439	3.0	133	0.93	0.87
1L-23	2525	2409	0.043	0.078	1130.5	5.6	463	3.2	141	0.99	0.92

Table C.7- Unit strain gage data, Waverly, Missouri test site (Metric Units)

Load	O-cell Load		Upwrd	Bottom	Level 2	O-cell to	Level 3	SG-2 to	Level 4	SG-3 to	SG-4 to
Test	Gross	Net	Mvment	of Cell		SC-2		SC-3		SG-4	0 Shear
Incre.					Avg.	171.8 to	Avg.	175.1 to	Avg.	178.2 to	182.7 to
					Load	175.1	Load	178.2	Load	182.7	184.9
	(MN)	(MN)	(mm)	(mm)	(MN)	(kPa)	(MN)	(kPa)	(MN)	(kPa)	(kPa)
1L-0	0	0	0	0	0	0	0.00	0	0.00	0	0
1L-1	1.06	0.04	0.1	0.1	0.58	-32.4	0.26	10.9	0.07	0.5	-2.3
1L-2	2.04	1.01	0.1	0.1	0.76	5.7	0.33	16.3	0.09	2.4	-0.6
1L-3	3.01	1.99	0.1	0.2	0.98	42.0	0.42	23.3	0.12	4.2	1.8
1L-4	3.98	2.96	0.1	0.2	1.28	74.0	0.53	33.4	0.16	6.7	4.8
1L-5	4.96	3.93	0.2	0.3	1.71	100.0	0.71	46.8	0.20	11.6	7.9
1L-6	5.93	4.90	0.2	0.4	2.10	128.4	0.87	58.3	0.24	15.8	11.3
1L-7	6.90	5.87	0.3	0.5	2.53	154.0	1.04	72.6	0.29	19.9	15.0
1L-8	7.87	6.85	0.3	0.5	2.94	181.3	1.21	84.6	0.34	24.6	18.4
1L-9	8.84	7.82	0.4	0.6	3.33	208.8	1.37	97.2	0.39	28.3	22.2
1L-10	9.82	8.79	0.4	0.7	3.78	234.2	1.54	111.8	0.44	32.6	25.9
1L-11	10.79	9.77	0.5	0.8	4.21	260.2	1.71	125.6	0.50	36.2	31.0
1L-12	11.76	10.73	0.5	0.8	4.62	287.0	1.87	138.5	0.54	40.7	33.7
1L-13	12.73	11.71	0.5	0.9	5.06	312.6	2.05	152.1	0.60	44.8	38.8
1L-14	13.71	12.68	0.6	1.0	5.51	337.6	2.24	166.2	0.65	49.8	42.2
1L-15	14.68	13.65	0.6	1.1	5.99	361.0	2.44	181.0	0.73	54.0	48.4
1L-16	15.65	14.63	0.7	1.2	6.49	383.6	2.65	196.7	0.78	59.3	52.4
1L-17	16.63	15.60	0.8	1.3	6.97	407.3	2.85	211.5	0.85	64.0	57.5
1L-18	17.60	16.57	0.8	1.4	7.50	428.6	3.06	227.9	0.91	69.3	62.3
1L-19	18.57	17.54	0.9	1.6	8.37	433.6	3.42	254.9	1.02	78.2	70.5
1L-20	19.54	18.52	0.9	1.6	8.53	472.6	3.49	259.6	1.05	79.6	72.8
1L-21	20.51	19.49	1.0	1.7	9.03	495.2	3.70	275.0	1.12	84.5	77.9
1L-22	21.49	20.46	1.1	1.8	9.52	518.2	3.91	290.1	1.18	89.5	83.0
1L-23	22.46	21.43	1.1	2.0	10.06	539.1	4.11	307.2	1.25	94.5	88.1

Table C.8- Unconfined compressive strength of rock cores Piers 9 and 10

	Pier 9				
	Elevation (ft)				
	600-592	592-585	585-570	570-560	560-550
	(tsf)	(tsf)	(tsf)	(tsf)	(tsf)
	44.0	27	59.4	52	56.5
	20.9	63	101.9	14.4	27.5
	2.3		80.2	41.1	23.9
	6.7		35.3	55.4	24.8
	1.9		39.8	23.1	
	9.0		33.7	*117.4	
	2.4		89.5	*160.9	
	1.8		147.9		
	2.5		122		
			16.6		
			60.2		
			56.4		
			52.4		
			68		
Mean	10.2	45.0	68.8	37.2	33.2
Std. Dev	14.1	25.5	36.4	17.9	15.6
	Pier 10				
	Elevation (ft)				
	602-594	594-586	586-570	570-560	560-550
	6.1	55.9	25.5	*128.8	78.8
	4.6	39.3	102.1	14.0	62.8
	2.9		137.6	26.0	17.7
	8.2		183.2	33.3	74.6
	3.6		262.7	35.7	120.2
			148.1	18.6	
			144.1	24.5	
			70.4		
			233		
			68.6		
			38.4		
Mean	5.1	47.6	128.5	25.4	70.8
Std. Dev	2.1	11.7	76.8	8.3	36.8

* Values not used in calculation of mean

Table C.9- Unconfined compressive strength of rock cores Piers 11 and 12

	Pier 11				
	Elevation (ft)				
	Weir (A)	Weir (B)	Weir (C)	Weir (D)	Weir (E)
	609-600	600-584.7	584.7-574	574-564	564-555
	(tsf)	(tsf)	(tsf)	(tsf)	(tsf)
	1.6	3.1	9.2	2.1	56.7
	0.8*	2.1	13.9	7.2	78.8**
	4.1	0.7*	38.1	9.1	62.8**
	4.1	5.8	37.2	10.4	17.7**
	4.1	3.1		8.1	74.6**
	3.6	9.1		18.8	120.2**
	4.7	10.2		7.5	
	5.3	4.0		11.1	
	7.2	4.3			
	5.6	2.9			
		7.9			
		3.3			
		3.4			
Mean	4.5	4.9	24.6	9.3	68.5
Std. Dev	1.5	2.7	15.2	4.7	33.4
	Pier 12				
	Elevation (ft)				
	631-602	631-602	602-574		
	2.2	2.2	101.9		
	10.9	2.1	86.7		
	72.7	12.0	77.8		
	7.3	13.7	194.0		
	13.3	4.5	35.4		
	12.1	25.2	2.2		
	2.7	13.7	7.8		
	11.2	3.7	170.9		
	9.2	12.2	7.0		
	19.8	4.6	20.2		
	7.6	3.0	136.1		
	10.4	8.2	66.5		
			31.1		
			36.2		
Mean		11.9	69.6		
Std. Dev		14.2	62.1		

* Values not used in calculation of mean

** Data from Pier 10

Table C.10- Unconfined compressive strength and SPT Data for Waverly, Mo.
(Elevation 633.0 to 606.4 ft.).

Station	Offset	Elev.	Unconfined Compressive Strength & SPT Data			
			Pier 9	Pier 10	Pier 11	Pier 12
	(ft)	(ft)	(tsf)	(tsf)	(tsf)	(tsf)
100+50	24' Lt.	632.9				9.2
100+20	42' Lt.	632.9				10.3
100+10	24" Lt.	631.3				10-30-38
100+50	24' Lt.	629.4				100 in 13"
100+22	60' Lt.	629.3				2.2
100+50	24' Lt.	628.2				10.9
100+10	24' Lt.	626.9				72.7
100+20	42' Lt	626.0				7.3
100+10	60" Lt	625.8				13.3
100+22	60' Lt	624.2				12.1
100+50	24' Lt.	622.8				2.7
100+10	24' Lt.	622.3				11.2
100+10	60'Lt.	620.5				9.2
100+22	60' Lt.	618.2				19.8
100+20	42' Lt.	616.8				7.6
100+10	24' Lt.	616.3				10.4
100+50	24' Lt.	616.0				2.2
100+10	60' Lt.	614.9				2.1
100+50	24' Lt.	613.5				100 in 4"
100+10	24' Lt.	612.4				12
100+50	24' Lt.	612.1				13.7
100+20	42' Lt.	611.0				4.5
100+10	60' Lt.	610.9				25.2
100+22	60' Lt.	609.8				13.7
95+60.8	56.6'Lt.	609.8			12-28-42	
95+64	35.5' Lt.	609.2			100 in 11.5"	
					Weir (A)	
95+60.4	30.1'Lt.	608.4			1.6	
100+50	24' Lt.	607.6				3.7
95+77	28' Lt.	607.4			0.8	
95+78.5	55.3' Lt.	607.2			4.1	
100+20	42' Lt.	606.6				12.2
100+10	24' Lt.	606.4				4.6

* Values not used in calculation of mean

Table C.11- Unconfined compressive strength and SPT Data for Waverly, Mo. (Elevation 604.3 to 598.2 ft.).

Station	Offset	Elev.	Unconfined Compressive Strength & SPT Data			
			Pier 9	Pier 10	Pier 11	Pier 12
	(ft)	(ft)	(tsf)	(tsf)	(tsf)	(tsf)
91+53.1	54.7'Lt.	604.3		100 in 8"		
100+10	60' Lt.	604.0				3
91+45.8	40.8' Lt.	604.0		100 in 10"		
95+64	35.5' Lt.	603.8			4.1	
95+60.8	56.6' Lt.	603.8			4.1	
100+22	60' Lt.	603.4				8.2
91+36.5	52.2' Lt.	603.2		8.9		
89+02	28' Lt.	602.9	100 in 9"			
88+95	42' Lt.	602.9	74 in 6"			
95+77	28' Lt.	602.8			3.6	
88+88	28' Lt.	602.5	100 in 11"			
95+78.5	53.3' Lt.	601.8			4.7	
95+60.4	30.1' Lt.	601.8			5.3	
100+50	24' Lt.	601.5				101.9
89+02	56' Lt.	601.5	5.5			
100+10	24' Lt.	601.3				86.7
91+53.1	54.7' Lt.	601.1		6.1		
100+20	42' Lt.	601.0				77.8
88+88	28' Lt.	600.9	9.5			
95+64	35.5' Lt.	600.8			7.2	
95+60.8	56.6' Lt.	600.8			5.6	
88+88	91' Lt.	600.4	100 in 6"			
100+22	60' Lt.	600.0				194.0
					Weir (B)	
95+78.5	55.3' Lt.	600.0			100 in 10"	
91+38	28' Lt.	599.7		4.6		
100+10	60' Lt.	599.5				35.4
91+50	32' Lt.	599.2		2.9		
88+95	42' Lt.	599.2	44.0			
95+77	28' Lt.	599.1			3.1	
91+38	28' Lt.	599.1		100 in 9"		
100+50	24' Lt.	598.2				100 in 5"
95+78.5	55.3' Lt.	598.2			2.1	

* Values not used in calculation of mean

Table C.12- Unconfined compressive strength and SPT Data for Waverly, Mo.
(Elevation 598.1 to 587.1 ft.).

Station	Offset	Elev.	Unconfined Compressive Strength & SPT Data			
			Pier 9	Pier 10	Pier 11	Pier 12
	(ft)	(ft)	(tsf)	(tsf)	(tsf)	(tsf)
89+02	28' Lt.	598.1	20.9			
95+60.4	30.1' Lt.	597.4			0.7	
89+02	56' Lt.	597.1	100 in 2"			
91+36.5	52.2' Lt.	596.9	2.3			
100+50	24' Lt.	596.6				2.2
91+45.8	40.8' Lt.	596.4		8.2		
88+88	91' Lt.	596.0	6.7			
91+38	28' Lt.	595.5		3.6		
89+02	56' Lt.	595.0	1.9			
100+50	24' Lt.	594.6				7.8
95+64	35.5' Lt.	594.2			5.8	
95+60.8	56.6' Lt.	594.2			3.1	
88+88	28' Lt.	594.2	9.0			
95+60.4	30.1' Lt.	594.0			9.1	
88+95	42' Lt.	593.3	2.4			
88+88	91' Lt.	593.2	1.8			
95+78.5	55.3' Lt.	592.8			10.2	
89+02	28' Lt.	592.8	2.5			
95+60.8	56.6' Lt.	592.2			4.0	
91+45.8	40.8' Lt.	591.9		55.9		
91+50	32' Lt.	591.7		39.3		
100+10	24' Lt.	591.4				170.9
95+64	35.5' Lt.	591.2			4.3	
95+77	28' Lt.	590.6			100 in 7.5"	
91+36.5	52.2' Lt.	590.2		100 in 4"		
95+60.4	30.1' Lt.	590.1			100 in 5.5"	
89+02	56' Lt.	589.3	27			
88+95	42' Lt.	589.2	63			
91+50	32' Lt.	588.4		100 in 2.5"		
95+60.8	56.6' Lt.	588.3			100 in 5"	
100+50	24' Lt.	588.0				7
91+45.8	40.8' Lt.	588.0		100 in 2"		
95+64	35.5' Lt.	587.7			100 in 5"	
95+48.5	53.3' Lt.	587.2			2.9	
88+95	42' Lt.	587.1	100 in 3"			

* Values not used in calculation of mean

Table C.13- Unconfined compressive strength and SPT data for Waverly, Mo.
(Elevation 586.8 to 576.4 ft.).

Station	Offset	Elev.	Unconfined Compressive Strength & SPT Data			
			Pier 9	Pier 10	Pier 11	Pier 12
	(ft.)	(ft)	(tsf)	(tsf)	(tsf)	(tsf)
89+02	28' Lt.	586.8	100 in 3"			
95+60.4	30.1' Lt.	586.7			7.9	
100+10	24' Lt.	586.5				20.2
88+88	28' Lt.	586.5	100 in 3"			
95+77	28" Lt.	585.9			3.3	
91+36.5	52.2' Lt.	585.8		25.5		
91+50	32' Lt.	585.7		102.1		
88+88	91' Lt.	584.9	100 in 4"			
91+38	28' Lt.	584.8		137.6		
95+64	35.5' Lt.	584.7			3.4	
89+02	28' Lt.	584.4	59.4		Weir (C)	
88+95	42' Lt.	583.9	101.9			
95+60.8	56.6' Lt.	583.8			9.2	
88+88	28' Lt.	583.6	80.2			
88+88	91' Lt.	583.2	35.3			
95+77	28' Lt.	582.9			13.9	
100+50	24' Lt.	582.8				100 in 2"
100+50	24' Lt.	582.6				136.1
100+10	60' Lt.	582.6				66.5
95+60.8	56.6' Lt.	582.2			38.1	
89+02	56' Lt.	581.9	100 in 3"			
91+50	32' Lt.	581.8		183.2		
88+88	28' Lt.	581.3	100 in 2"			
88+88	28' Lt.	580.9	39.8			
89+02	56' Lt.	579.7	33.7			
88+95	42' Lt.	579.5	89.5			
88+88	91' Lt.	579.3	147.9			
95+78.5	55.3' Lt.	579.2			100 in 4"	
91+38	28' Lt.	578.8		262.7		
100+50	24' Lt.	578.7				31.1
89+02	28' Lt.	578.6	122.0			
91+50	32' Lt.	578.5		148.1		
95+60.4	30.1' Lt.	578.0			37.2	
91+36.5	52.2' Lt.	576.7		144.1		
91+53.1	54.7' Lt.	576.4		70.4		

* Values not used in calculation of mean

Table C.14- Unconfined compressive strength and SPT data for Waverly, Mo.
(Elevation 575.0 to 563.1 ft.).

Station	Offset	Elev.	Unconfined Compressive Strength & SPT Data			
			Pier 9	Pier 10	Pier 11	Pier 12
	(ft)	(ft)	(tsf)	(tsf)	(tsf)	(tsf)
89+02	56' Lt.	575.0	16.6			
89+02	28' Lt.	575.0	60.2			
95+77	28' Lt.	574.9			100 in 4"	
100+50	24' Lt	574.5				36.2
91+38	28' Lt	573.8		233	Weir (D)	
95+60.8	56.6' Lt	573.4			2.1	
88+95	42' Lt.	572.8	56.4			
88+88	28' Lt.	572.7	52.4			
88+88	91' Lt.	572.6	68			
88+95	42' Lt	571.9	74 in 2"			
89+02	28' Lt	571.6	74 in 3"			
91+36.5	52.2' Lt	571.2		68.6		
95+60.8	56.6' Lt	571.2			7.2	
91+53.1	54.7' Lt	570.9		38.4		
95+78.5	53.3' Lt	570.2			9.1	
88+88	28' Lt	569.8	52			
89+02	28' Lt	569.7	14.4			
95+64	35.5' Lt	569.6			10.4	
91+38	28' Lt	569.3		128.8		
95+77	28' Lt	568.4			8.1	
95+60.8	56.6' Lt	568.3			100 in 3"	
88+88	91' Lt	567.8	41.1			
91+36.5	52.2' Lt	567.7		14		
100+50	24' Lt	567.6				100 in 5"
95+64	35.5' Lt	567.3			100 in 4"	
89+02	56' Lt	566.7	74 in 3"			
95+60.4	30.1' Lt	566.5			18.8	
91+38	28' Lt	565.3		26		
95+60.8	56.6' Lt	565.2			7.5	
95+64	35.5' Lt	565.1			11.1	
95+60.4	30.1' Lt	564.5			100 in 2"	
91+36.5	52.2' Lt	563.7		33.3	Weir (E)	
88+88	91' Lt	563.3	55.4			
100+50	24' Lt	563.2				33.2
91+45.8	40.8' Lt	563.1		35.7		

* Values not used in calculation of mean

Table C.15- Unconfined compressive strength and SPT Data for Waverly, Mo.
(Elevation 562.6 to 542.2 ft.).

Station	Offset	Elev.	Unconfined Compressive Strength & SPT Data			
			Pier 9	Pier 10	Pier 11	Pier 12
	(ft)	(ft)	(tsf)	(tsf)	(tsf)	(tsf)
89+02	56' Lt	562.6	23.1			
91+50	32' Lt	562.4		18.6		
91+38	28' Lt	562.3		24.5		
95+78.5	53.3' Lt	561.7			56.7	
88+88	28' Lt.	561.7	117.4*			
89+02	56' Lt	560.2	160.9*			
91+50	32' Lt	559.4		78.8		
88+88	91' Lt	558.4	56.5			
91+36.5	52.2' Lt	557.8		62.8		
88+88	28' Lt	557.1	27.5			
91+50	32' Lt	556.4		17.7		
91+38	28' Lt	555.8		74.6		
91+53.1	54.7'	555.6		120.2		
89+02	56'	552.7	100 in 3"			
88+88	91'	552.6	23.9			
100+50	24'	552.2				100 in 3"
88+88	28'	551.7	24.8			
89+02	28'	551.4	74 in 5"			
88+88	28'	551.2	100 in 4"			
100+50	24' Lt	542.2				100 in 2"

* Values not used in calculation of mean

APPENDIX D
SI UNITS FOR CHAPTER SEVEN TABLES AND FIGURES

Table D.1- Summary of measured unit side shear values and average unconfined compressive strength (q_u) values of shale at test sites.

Formation	Elevation of Strata	Shaft Section used to calc unit side shear	Elevation of Shaft Segment	q_u Avg.	Std. Dev.	Measured unit side shear, f_s [1]
	(m)		(m)	(kPa)	(kPa)	(kPa)
		Lexington				
Bevier (C1)	180.4 - 187	TS-2 upper cell to TOS	180.2 – 184.4	3811	2210	1020
Bevier (C1)	180.4 - 187	TS-2 SG-3 to SG-4	181.7 – 182.7	3811	2210	>694 [2]
Bevier (C1)	180.4 - 187	TS-2 upper cell to SG-3	180.2 – 181.7	3811	2210	>1653
Bevier (C2)	176.4 – 180.4	TS-2, stage 1, lower cell to SG-2	176.2 – 178.7	3001	2565	885
Verdigris (D)	173.5 – 176.4	TS-1A, SG-4 to TOS	174.5 – 175.9	1212	1244	>391
Verdigris (D)	173.5 – 176.4	TS-1A, SG-3 to SG-4	173 – 174.5	1212	1244	>968
Croweburg (E)	169 – 173.5	TS-1A, O-cell to SG-2	170.5 -172	1716	1552	723
Croweburg (E)	169 – 173.5	TS-1A, SG-1 to O-cell	169.6 – 170.5	1716	1552	963
		Grandview				
W. Chanute	282.6 – 284.7	SG-6 to TOS	282.5 – 284.7	938	220	>301
Chanute	279.3 – 282.6	SG-5 to SG-6	280.7 - 282.5	690	335	295
Chanute	916.5 – 282.6	SG-4 to SG-5	279.3 – 280.7	690	335	453
Cement City	277.8 –279.3	SG-3 to SG-4	277.8 – 279.3	37000	17217	>1651
Quivira	275.8 – 277.8	O-cell to SG-3	275.8 – 277.8	1427	287	460 [3]
Westerville	273.7 – 275.8	SG-1 to O-cell	273.7 – 275.8	62900	29868	>2293
Wea	< 273.7	Tip to SG-1	272.3 – 273.7	2308	833	565
		Waverly				
Weir (A)	182.9 – 185.6	SG-4 to O shear	182.7 – 184.9	430	140	>88
Weir (B)	178.2 – 182.9	SG-3 to SG-4	178.2 – 182.7	470	260	>94
Weir (C)	174.9 – 178.2	SG-2 to SG-3	175.1 – 178.2	2360	1460	>306
Weir (D)	171.9 – 174.9	O-cell to SG-2	171.8 – 175.1	890	450	>587
Weir (E)	169.2 – 171.9			6560	3200	

(1) Values reported are ultimate values unless otherwise indicated.
(2) The symbol ">" indicates that the ultimate unit side shear was not reached during test, value reported is maximum value during test.
(3) Assumed ultimate unit side shear.

Table D.2- Summary of back-calculated alpha values of shale at Missouri test sites.

Formation	Shaft Section used to calc unit side shear	Elevation of shaft Segment	q_u Avg.	q_u Std. Dev.	Meas. Unit side shear, f_s[1]	α Avg..	α +1 Std Dev	α −1 Std. Dev.
		(m)	(kPa)	(kPa)	(kPa)			
	Lexington							
Bevier (C1)	TS-2 upper cell to TOS	180.2-184.4	3811	2210	1020	0.27	0.17	0.64
Bevier (C1)	TS-2 SG-3 to SG-4	181.7-182.7	3811	2210	>694[2]	0.18	0.12	0.44
Bevier (C1)	TS-2 upper cell to SG-3	180.2-181.7	3811	2210	>1653	0.43	0.28	1.04
Bevier (C2)	TS-2, stage 1, lower cell to SG-2	176.2-178.7	3001	2565	885	0.29	0.16	2.04
Verdigris (D)	TS-1A, SG-4 to TOS	174.5-175.9	1212	1244	>391	0.32	0.16	-13.7
Verdigris (D)	TS-1A, SG-3 to SG-4	173 – 174.5	1212	1244	>968	0.80	0.39	-33.7
Croweburg (E)	TS-1A, O-cell to SG-2	170.5 - 172	1716	1552	723	0.42	0.22	4.47
Croweburg (E)	TS-1A, SG-1 to O-cell	169.6–170.5	1716	1552	963	0.56	0.30	5.94
	Grandview							
W. Chanute	SG-6 to TOS	282.5-284.7	938	220	>301	0.33	0.26	0.43
Chanute	SG-5 to SG-6	280.7-282.5	690	335	295	0.43	0.29	0.84
Chanute	SG-4 to SG-5	279.3-280.7	690	335	453	0.67	0.45	1.30
Cement City	SG-3 to SG-4	277.8-279.3	37000	17217	>1651			
Quivira	O-cell to SG-3	275.8-277.8	1427	287	460[3]	0.32	0.27	0.40
Westerville	SG-1 to O-cell	273.7-275.8	62900	29868	>2293			
Wea	Tip to SG-1	272.3-273.7	2308	833	565	0.24	0.18	0.38
	Waverly							
Weir (A)	SG-4 to O shear	182.7-184.9	430	140	>88	0.20	0.15	0.30
Weir (B)	SG-3 to SG-4	178.2-182.7	470	260	>94	0.20	0.14	0.42
Weir (C)	SG-2 to SG-3	175.1-178.2	2360	1460	>306	0.13	0.08	0.34
Weir (D)	O-cell to SG-2	171.8-175.1	890	450	>587	0.67	0.44	1.35
Weir (E)			6560	3200				

(1) Values reported are ultimate values unless otherwise indicated.
(2) The symbol ">" indicates that the ultimate unit side shear was not reached during test, value reported is maximum value during test.
(3) Assumed ultimate unit side shear.

Table D.3- Comparison of measured and predicted unit side shear determined by various methods.

Formation	Shaft Segment	Elevation of Shaft Segment	q_u Avg.	Meas. unit side shear, f_s [1]	Predicted ultimate unit side shear				
					Horvath & Kenney (1979) $\alpha = 0.25$	Rowe & Armitage (1987) $\alpha = 0.45$	Reese & O'Neill (1988) $\alpha = 0.21$	Kulhawy & Phoon (1993) $\psi = 2$	O'Neill & Reese (1999) [4]
		(m)	(kPa)	(kPa)	(kPa)	(kPa)	(kPa)	(kPa)	(kPa)
Lexington									
Bevier C1	TS-2, upper cell to TOS	180.2 - 184.4	3811	1020	488.0	878.5	410.0	877.4	190.6
Bevier C1	TS-2, SG-3 to SG-4	181.7 - 182.7	3811	>694[2]	488.0	878.5	410.0	877.4	228.7
Bevier C1	TS-2, upper cell toSG-3	180.2 - 181.7	3811	>1653	488.0	878.5	410.0	877.4	228.7
Bevier C2	TS-2, stage 1, lower cell to SG-2	176.2 - 178.7	3001	885	433.1	779.6	363.8	778.6	270.1
Verdigris (D)	TS-1A SG-4 to TOS	174.5 - 175.9	1212	>391	275.2	495.4	333.3	494.8	169.7
Verdigris (D)	TS-1A SG-3 to SG-4	173.0 - 174.5	1212	>968	275.2	495.4	333.3	494.8	169.7
Croweburg (E)	TS-1A O-cell to SG-2	170.5 - 172.0	1716	723	327.5	589.5	471.9	588.1	240.2
Croweburg (E)	TS-1A SG-1 to O-cell	169.6 - 170.5	1716	963	327.5	589.5	471.9	588.1	240.2
Grandview									
W. Chanute	SG-6 to TOS	282.5 - 284.7	938	>301	242.4	436.3	258.5	435.8	56.4
Chanute	SG-5 to SG-6	280.7 - 282.5	690	295	207.7	373.8	189.8	373.3	55.2
Chanute	SG-4 to SG-5	279.3 - 280.7	690	453	207.7	373.8	189.8	373.3	69.0
Cement City	SG-3 to SG-4	277.8 - 279.3	37000						
Quivira	O-cell to SG-3	275.8 - 277.8	1427	460[3]	295.8	532.4	385.0	531.8	196.0
Westerville	SG-1 to O-cell	273.7 - 275.8	62900						
Wea	Tip to SG-1	272.3 - 273.7	2308	565	379.1	682.5	318.5	681.6	323.1
Waverly									
Weir (A)	SG-4 to O shear	182.7 - 184.9	430	>88	169.5	305.1	126.4	304.7	119.5
Weir (B)	SG-3 to SG-4	178.2 - 182.7	470	>94	171.2	308.2	129.0	307.9	140.8
Weir (C)	SG-2 to SG3	175.1 - 178.2	2360	>306	383.7	690.7	322.3	689.8	329.8
Weir (D)	O-cell to SG-2	171.8 - 175.1	890	>587	235.9	424.7	133.6	424.1	195.9
Weir (E)			6560						

(1) Values reported are ultimate values unless otherwise indicated.   (2) The symbol ">" indicates that the ultimate unit side shear was not reached during test, value reported is maximum during test.   (3) Assumed unit side shear   (4) Method not intended for slump less than 175 mm (7 in)

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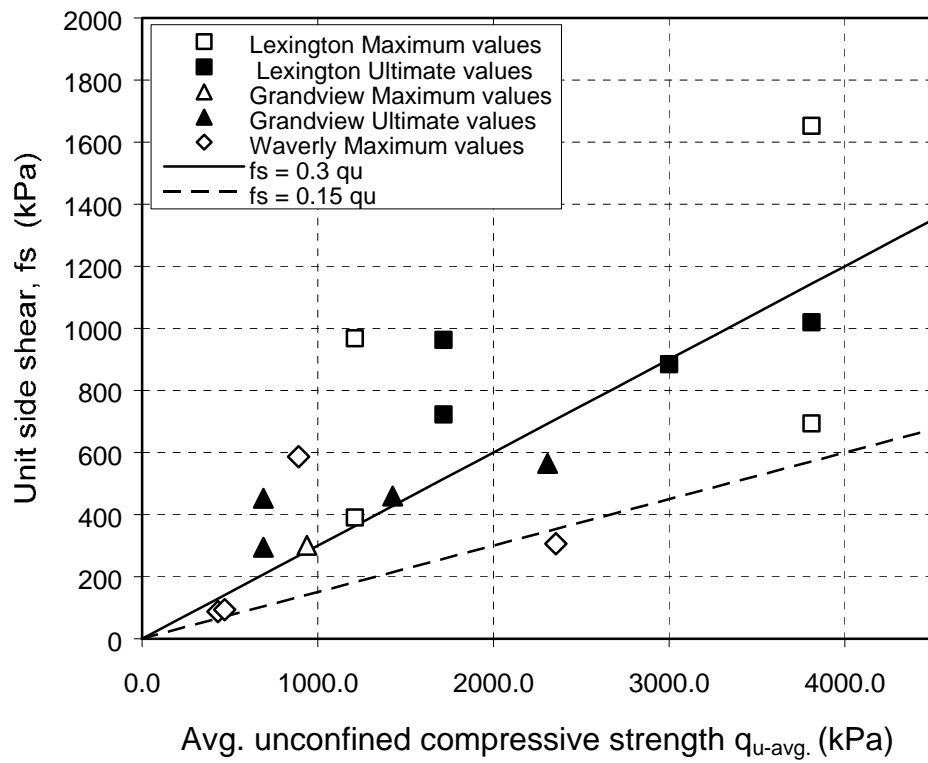
Figure D.1- Unit side shear versus average q_u.
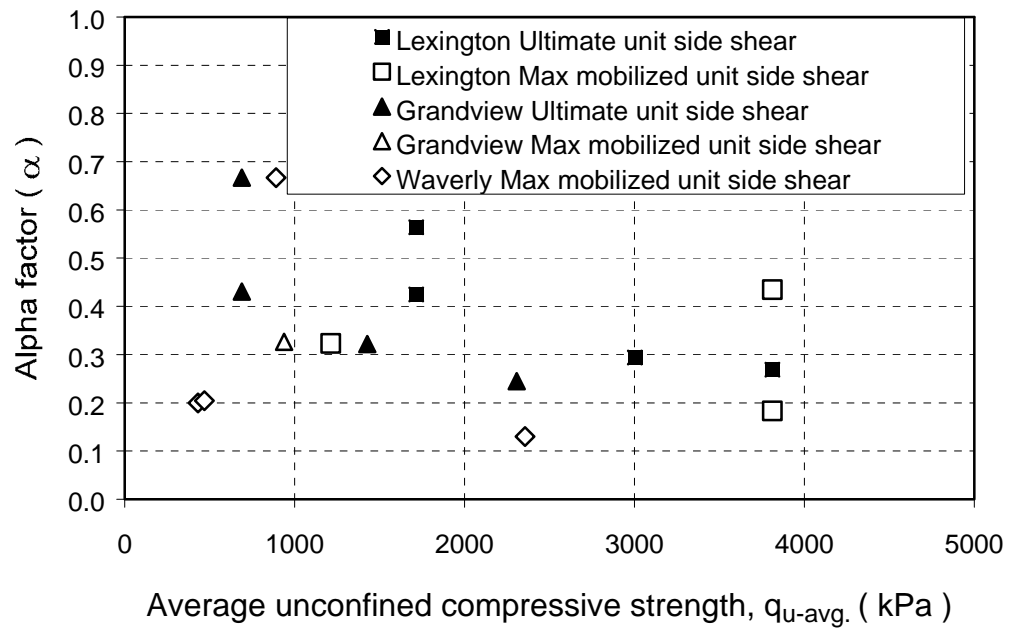
Figure D.2- Back-calculated alpha factor (α) versus average q_u for test sites in shale.
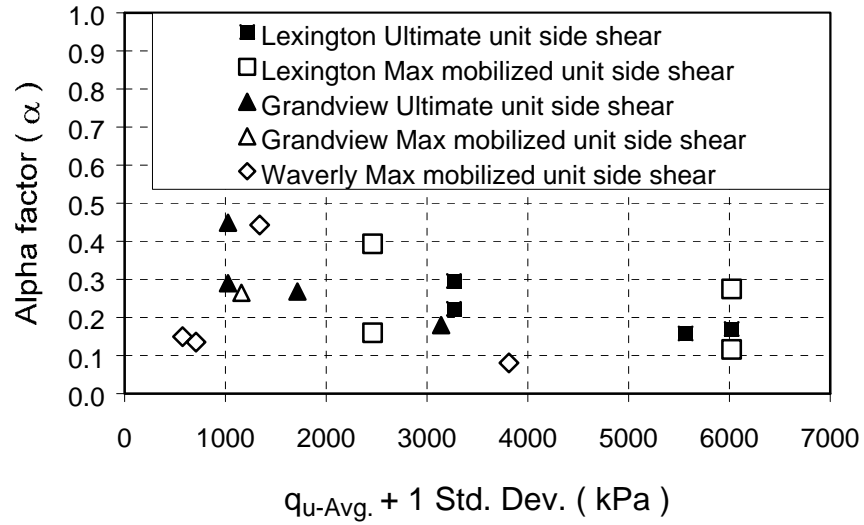
a. $q_{u\text{-avg.}}$ plus one standard deviation.
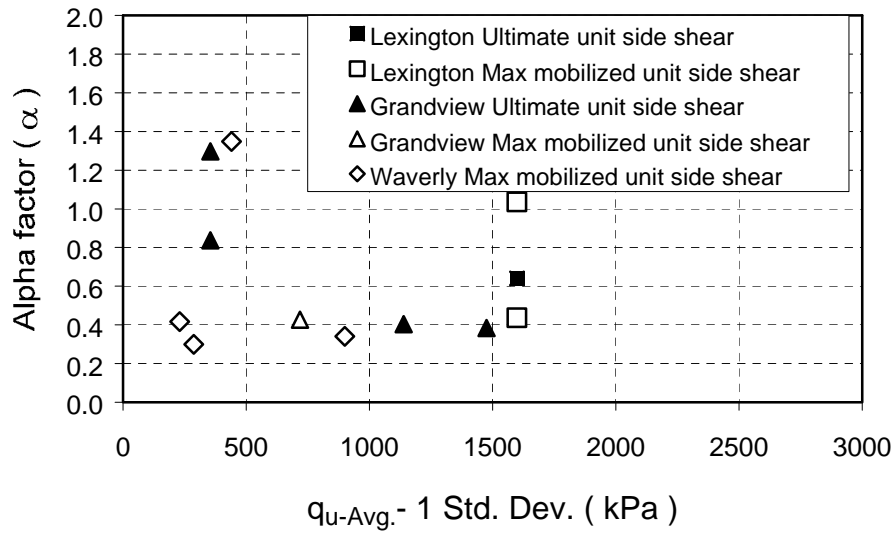
b. $q_{u\text{-avg.}}$ minus one standard deviation.

Figure D.3- Back-calculated alpha (α) versus q_u; (a) $q_{u\text{-avg.}}$ plus one standard deviation, (b) $q_{u\text{-avg.}}$ minus one standard deviation.

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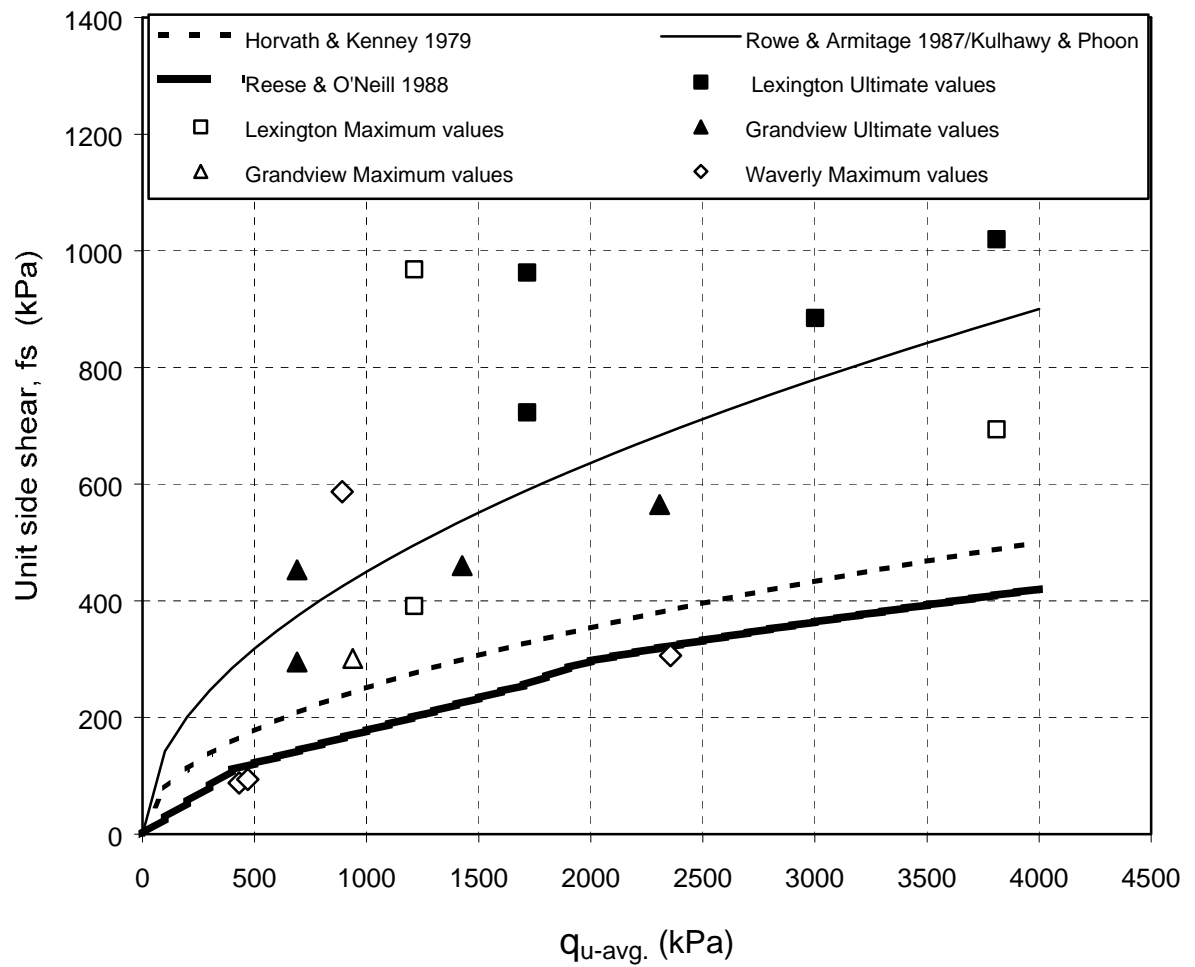
Figure D.4- Comparison of measured unit side shear data to predicted unit side shear by several methods.
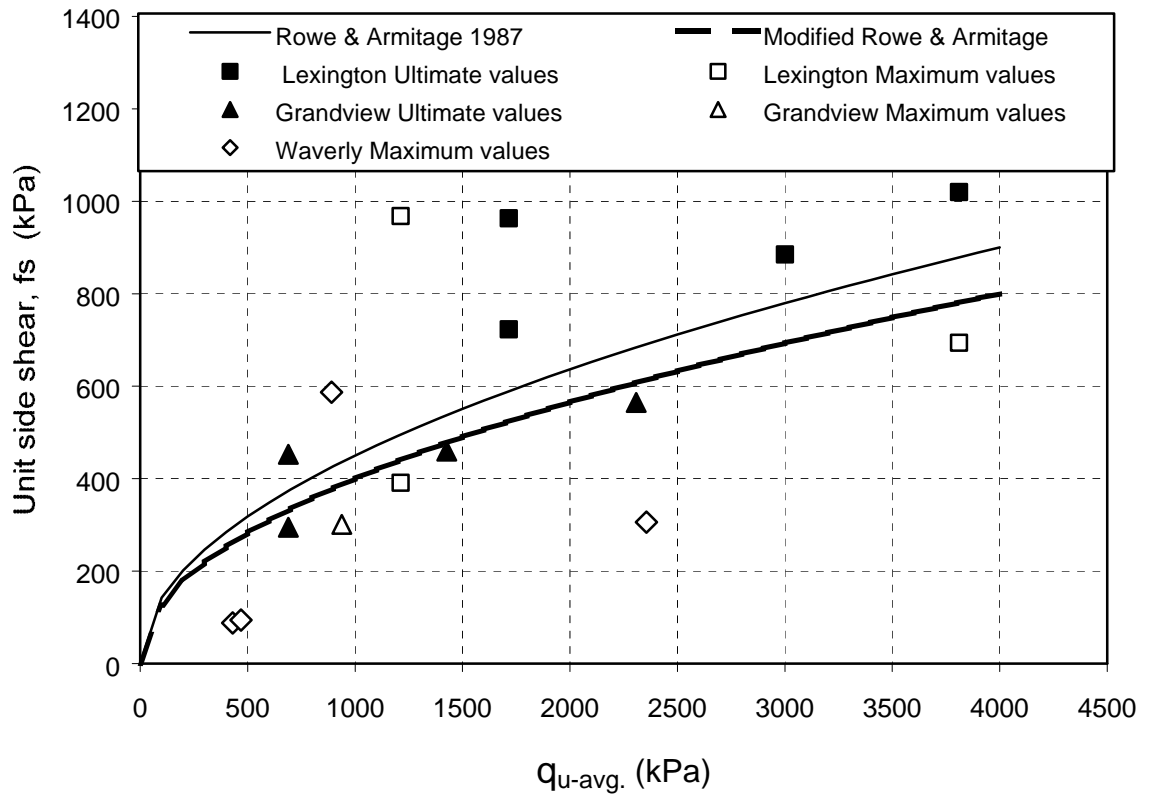
Figure D.6- Modified Rowe and Armitage method.
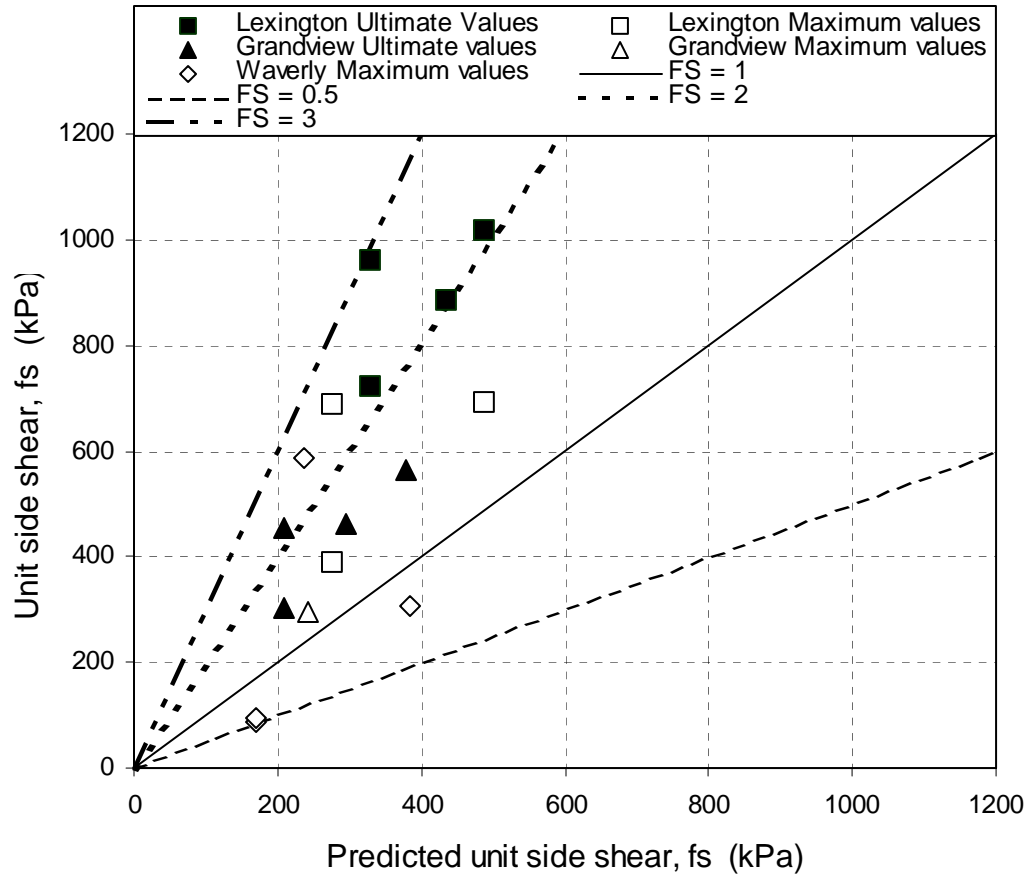
Figure D.7- Comparison of measured and predicted unit side shear value using the Horvath and Kenney (1979) method.
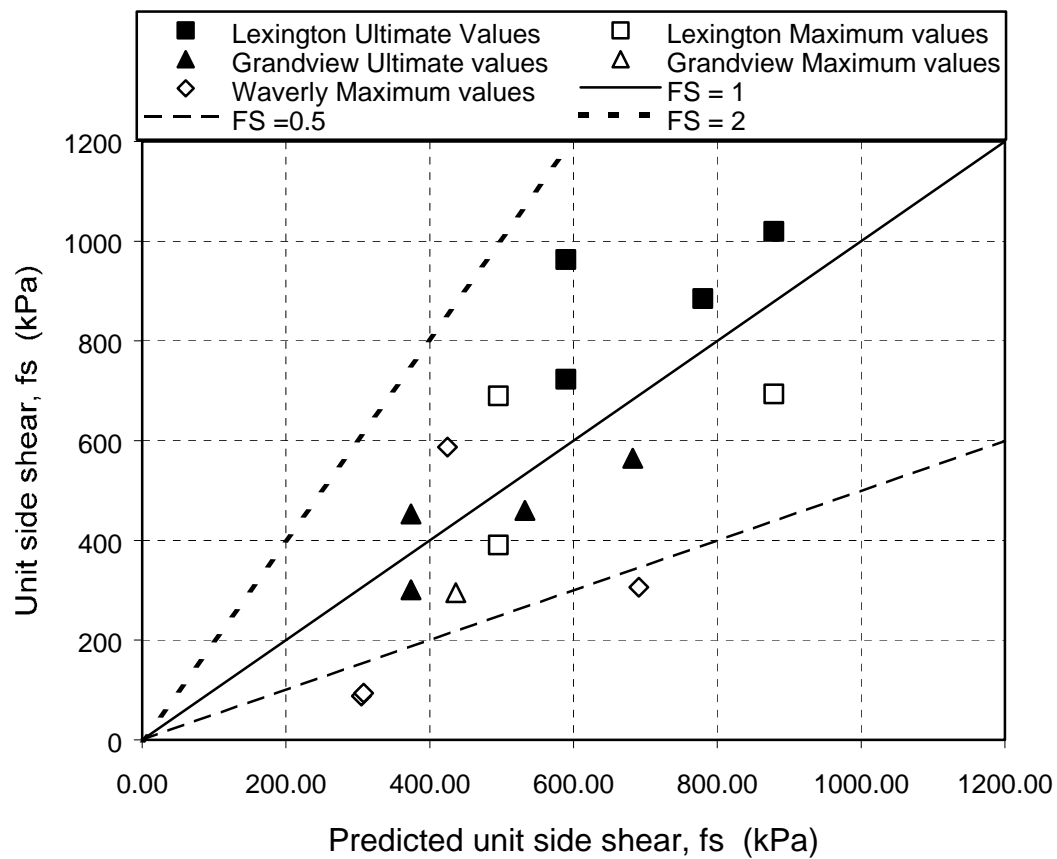
Figure D.8- Comparison of measured and predicted unit side shear using the Rowe and Armitage (1987) method.
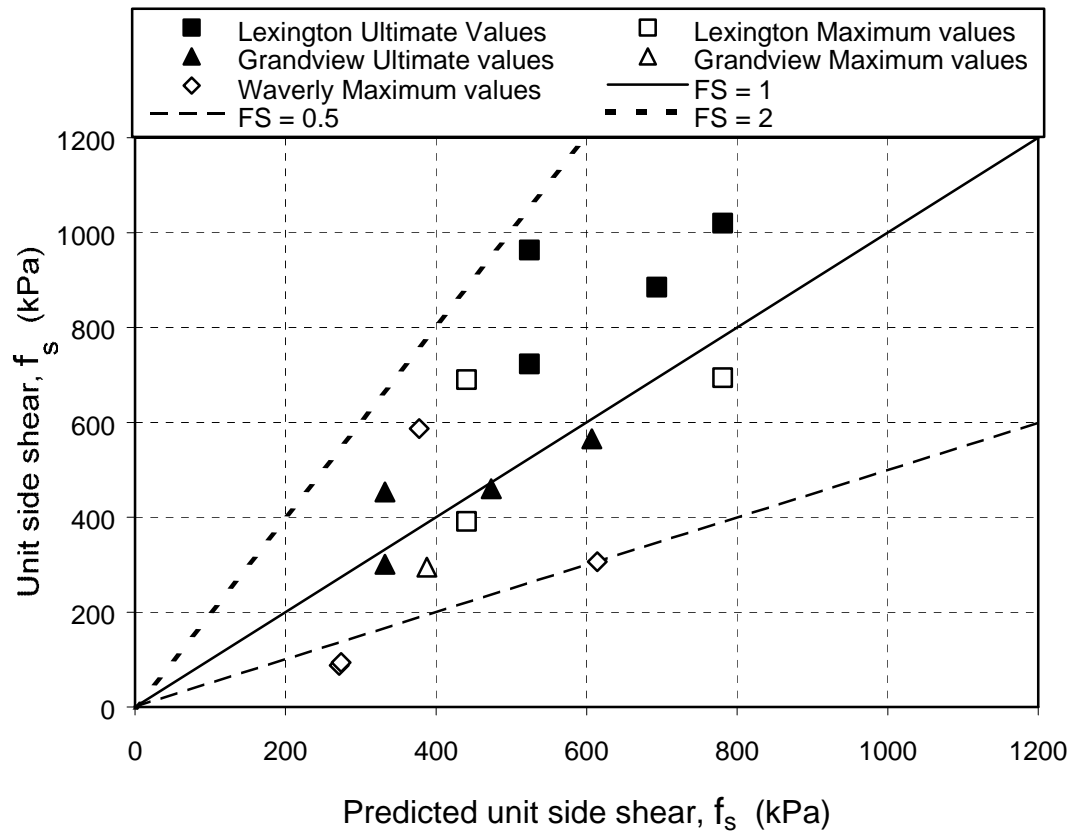
Figure D.9- Comparison of measured and predicted unit side shear using the modified
Rowe and Armitage (1987) method.

APPENDIX E
CONSTRUCTION AND LOAD TEST PHOTOGRAPHS FOR LEXINGTON
TEST SITE (HNTB 1999)

E1- View of platform, temporary casing, and deflection wall sheeting

E2- Test shaft ts-1A in foreground. Test shaft TS-2 in background.

E3- Drilling bucket and Manitowoc 4100 Series with drill assembly

E4- Hardened steel cutting edge at bottom of permanent casing

E5- Replacing slurry with water after seating permanent casing.

E6- Bullet tooth rock auger.

E7- Using rock auger to drill rock socket

E8- Cleaning hole by airlift method.

E9- Bottom of airlift pipe.

E10- Assembling sister bars, strain gages, vibrating wire
displacement transducers on carrying frame.

E11- Welding carrier frame to O-cell^{TM}

E12- Lifting O-cells and carrier frame

E13- Down hole rotating sonar "caliper" device

E14- Real-time sonar measurements and lowering of sonar.

E15- Airlift pipe and assembled load cells and carrier frames for TS-1A and TS-2

E16- Lowering O-cells and carrier frame into test shaft

E17- Lowering 125 mm tremie pipe to bottom of test shaft.

E18- Delivering concrete to test shaft by barge.

E19- Pumping concrete to tremie pipe.

E20- Dial gages for measuring displacement of top of carrying frame during load test.

E21- Water pump for pressurizing load cells.

E22- Recording readings from load test instruments.

MISSOURI DEPARTMENT OF TRANSPORTATION
Division of Materials

BORING DATA (CORE & SPT)

Sheet __12__ of __18__

Job No.	J4P1102						
County	Lafayette/Ray		**Route**	13		**Design**	A5664
Over	Missouri River					**Skew**	Right Angles
Logged by	Davis/Stevens					**Operator**	Lamberson/Wilde
Equipment	Failing 1500					**Drillers Hole No.**	A-96-55
Hole Stab. by	Casing					**Date of Work**	10/24/96

Bent	Station	Location	Barge Deck Elevation		LOG OF MATERIALS *	
22	0+112.7	10m LT.	209.39	0.0-1.54m	Barge.	
B-13 Formerly	11+244	10m LT.	209.39	1.54-8.50m	Water.	
TEST DATA				8.50-12.00m	Medium and fine grained sand, dense.	
Depth, m	**SPT Blows/15cm**	**Pocket Pen., kg/cm²**	**Est. Equiv. Qu, kPa**	12.00-15.84m	Coarse sand and fine gravel, medium dense.	
9.35	9-6-9	Sand		15.84-16.10m	Large cobble, hard.	
12.50	3-4-4	Sand		16.10-17.15m	Fine to medium grained, medium to thick bedded, arkosic	
15.50	5-13-12	Sand			and micaceous sandstone, hard, some cross bedding.	
19.30	50 in 8cm	9.0+		17.15-22.20m	Gray thinly laminated clay shale, moderately hard.	
23.80	50 in 6cm	Crumbled		22.20-22.30m	Black shale to coal.	
28.36	50 in 6cm	7.50		22.30-28.56m	Gray thinly cross-laminated fine grained calcareous,	
					micaceous silt shale, hard.	
				28.56-31.42m	Gray, well cemented, thinly laminated clay shale,	
					moderately hard.	

CORING LOG (NX Double Tube Barrel)								
From	**To**	**Run**	**Rec**	**Loss**	**% RQD**	**Notes**		
16.30	17.80	1.50	1.46	0.04	0.00			
17.80	19.30	1.50	1.50	0.00	0.00			
19.38	20.88	1.50	1.30	0.20	0.00			
20.88	22.30	1.42	1.42	0.00	0.00			
22.30	23.80	1.50	1.50	0.00	0.00			
23.86	25.36	1.50	1.48	0.02	0.00			
25.36	26.86	1.50	1.50	0.00	0.00			
26.86	28.36	1.50	1.50	0.00	0.00			
28.42	29.92	1.50	1.41	0.09	0.00			
29.92	31.42	1.50	1.38	0.12	0.00			

· Persons using this information are cautioned that the materials shown are determined by the equipment noted and accuracy of the "log of materials" is limited thereby and by judgment of the operator. THIS INFORMATION IS FOR DESIGN PURPOSES ONLY:

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MISSOURI DEPARTMENT OF TRANSPORTATION
Division of Materials

BORING DATA (CORE & SPT)

Job No.	J4P1102						
County	Lafayette/Ray		**Route**	13	**Design**	A5664	
Over	Missouri River				**Skew**	Right Angles	
Logged by	Davis/Stevens				**Operator**	Lamberson/Wilde	
Equipment	Failing 1500				**Drillers Hole No.**	A-96-55	
Hole Stab. by	Casing				**Date of Work**	10/24/96	

Bent	**Boring**	**Station**	**Location**	**Barge Deck Elevation**
22	B-13	0+112.7	10m LT.	209.39
Formerly	B-13	11+244	10m LT.	209.39

SOIL CLASSIFICATION TEST DATA					UNCONFINED COMPRESSIVE DATA		
Depth, m	**LL**	**PI**	**ASTM Class**		**Depth, m**	**Qu, kPa**	**P.P., kg/cm²**
9.35		NP	SP		16.70	4807	9.0+
					17.90	1381	9.0+
					20.30	949	9.0+
					21.50	637	9.0+
					23.70	1320	9.0+
					24.40	2473	9.0+
					26.20	2101	9.0+
					28.10	2494	9.0+
					28.70	2578	9.0+
					31.0	2213	9.0+

SIEVE ANALYSIS (Percent Passing)
AASHTO T88

Depth, m

	9.35							
19mm	100.00							
9.5mm	99.00							
4.75mm	97.00							
2.00mm	95.00							
.850mm	91.00							
.425mm	64.00							
.300mm	32.00							
.150mm	23.00							
.075mm	4.00							

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Boring B-13 0+112.7 10m LT.
Bent 22 11/04/96

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MISSOURI DEPARTMENT OF TRANSPORTATION
Division of Materials

BORING DATA (CORE & SPT)

Job No.	J4P1102						
County	Lafayette/Ray		Route	13	Design	A5664	
Over	Missouri River				Skew	Right Angles	
Logged by	Dietiker/Davis				Operator	Dodds/Lamberson	
Equipment	Failing 1500				Drillers Hole No.	A-96-54	
Hole Stab. by	Casing				Date of Work	10/24/96	

Bent	Station	Location	Barge Deck Elevation		LOG OF MATERIALS *	
22	0+122.7	10m RT.	209.27	0.0-1.52m	Barge.	
B-15 Formerly	11+254	10m RT.	209.27	1.52-7.52m	Water.	
TEST DATA				7.52-12.20m	Gray fine to medium grained sand with scattered gravel,	
Depth, m	SPT Blows/15cm	Pocket Pen., kg/cm²			medium dense.	
11.00	5-6-6	Sand		12.20-15.16m	Gray medium grained sand with scattered coarse grained sand,	
14.00	6-6-6	Sand			medium dense.	
17.00	50 in 6cm	Sand		15.16-15.40m	Coarse gravel and scattered cobbles.	
20.33	50 in 11cm	Sand		15.40-16.07m	Gray fine to medium grained sand, dense.	
23.44	50 in 7cm	Sand		16.07-17.07m	Brown to gray coarse grained sandstone, very hard,	
26.51	50 in 5cm	Sand			cut with rockbit.	
				17.07-21.94m	Gray clay shale, thinly laminated, moderately hard.	
				21.94-22.17m	Black coal seam.	
				22.17-28.71m	Gray thin to medium laminated, calcareous, micaceous	
					silt shale, moderately hard.	
				28.71-32.01m	Gray thinly laminated clay shale, moderately hard.	
				32.01-32.06m	Light brownish-gray, thin bedded, fine grained limestone,	
					very hard.	
				32.06-32.61m	Gray clay shale, moderately hard.	

CORING LOG (NX Double Tube Barrel)

From	To	Run	Rec	Loss	% RQD	Notes		
17.33	18.83	1.50	1.50	0.00	0.00	Shale		
18.83	20.33	1.50	1.18	0.32	0.00	Shale		
20.44	21.94	1.50	1.30	0.20	0.00	Shale		
21.94	23.44	1.50	1.50	0.00	0.00	Shale		
23.51	25.01	1.50	1.50	0.00	0.00	Shale		
25.01	26.51	1.50	1.50	0.00	0.00	Shale		
26.56	28.06	1.50	1.50	0.00	0.00	Shale		
28.06	29.56	1.50	1.34	0.16	0.00	Shale		
29.61	31.11	1.50	1.50	0.00	0.00	Shale		
31.11	32.61	1.50	1.50	0.00	0.00	Shale		

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MISSOURI DEPARTMENT OF TRANSPORTATION
Division of Materials

BORING DATA (CORE & SPT)

Job No.	J4P1102					
County	Lafayette/Ray		**Route**	13	**Design**	A5664
Over	Missouri River				**Skew**	Right Angles
Logged by	Dietiker/Davis				**Operator**	Dodds/Lamberson
Equipment	Failing 1500				**Drillers Hole No.**	A-96-54
Hole Stab. by	Casing				**Date of Work**	10/24/96

Bent	**Boring**	**Station**	**Location**	**Barge Deck Elevation**
22	B-15	0+122.7	10m RT.	209.27
Formerly	B-15	11+254	10m RT.	209.27

						UNCONFINED COMPRESSIVE DATA		
						Depth, m	**Qu, kPa**	**P.P., kg/cm²**
						18.20	720	9.0+
						19.60	1141	9.0+
						21.40	535	9.0+
						22.40	5759	9.0+
						24.50	1648	9.0+
						25.60	2097	9.0+
						29.0	2468	9.0+
						30.30	1579	9.0+
						31.80	2332	9.0+

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Boring B-15   0+122.7 10m RT.
Bent 22   11/04/95

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MISSOURI DEPARTMENT OF TRANSPORTATION
Division of Materials

BORING DATA (CORE & SPT)

Job No.	J4P1102						
County	Lafayette/Ray		Route	13	Design	A5664	
Over	Missouri River				Skew	Right Angles	
Logged by	Davis/Stevens/Dietiker				Operator	Lamberson/Wilde/Dodds	
Equipment	Failing 1500				Drillers Hole No.	A-96-53	
Hole Stab. by	Casing				Date of Work	10/23/96	

Bent	Station	Location	Barge Deck Elevation	LOG OF MATERIALS *	
22	0+117.7	C/L	209.16	0.0-1.55m	Barge.
B-14 Formerly	11+249	C/L	209.16	1.55-7.52m	Water.

TEST DATA

Depth, m	SPT Blows/15cm	Pocket Pen., kg/cm²	Est. Equiv. Qu, kPa		
7.52	14-25-30			7.52-9.00m	Gray fine and medium grained coarse sand, dense to very dense.
9.50	5-3-6			9.00-13.20m	Gray fine to medium grained sand and fine gravel with some
12.50	9-5-7				coarse grained sand, medium dense.
15.50	50 in 8cm			13.20-14.30m	Coarse sand and cobbles.
18.02	100 in 13cm	9.0+	570	14.30-14.75m	Coarse sand, medium dense.
22.66	50 in 8cm	9.0+		14.75-14.90m	Boulder, very hard.
27.24	50 in 6cm	7.0		14.90-15.90m	Fine sand and silt with some clay, very dense.
31.80	50 in 10cm			15.90-16.90m	Fine to medium grained sandstone, very hard, cut with rockbit.
				16.90-21.56m	Gray thinly laminated clay shale, moderately hard, cut with
					rockbit to 18.16m.
				21.56-21.76m	Black carbonaceous shale, moderately hard.
				21.76-21.98m	Black coal bed, hard.
				21.98-22.21m	Gray clay shale, soft, (underclay).
				22.21-22.39m	Dark gray shaly limestone to siltstone, very hard.
				22.39-28.89m	Gray thin to medium laminated, fine grained, calcareous,
					micaceous silt shale, well cemented, hard.

CORING LOG (NX Double Tube Barrel)

From	To	Run	Rec	Loss	% RQD	Notes		
18.16	19.66	1.50	1.50	0.00	0.00		28.89-32.71m	Gray clay shale, thinly laminated, moderately hard.
19.66	21.16	1.50	1.50	0.00	0.00		32.71-32.94m	Black coal seam, hard.
21.16	22.66	1.50	1.50	0.00	0.00		32.94-33.40m	Dark gray clay shale, poorly laminated, soft.
22.74	24.24	1.50	1.50	0.00	0.00			
24.24	25.74	1.50	1.50	0.00	0.00			
25.74	27.24	1.50	1.50	0.00	0.00			
27.30	28.80	1.50	1.49	0.01	0.00			
28.80	30.30	1.50	1.46	0.04	0.00			
30.30	31.80	1.50	1.50	0.00	0.00			
31.90	33.40	1.50	1.50	0.00	0.00			

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MISSOURI DEPARTMENT OF TRANSPORTATION
Division of Materials

BORING DATA (CORE & SPT)

Job No.	J4P1102					
County	Lafayette/Ray		**Route**	13	**Design**	A5664
Over	Missouri River				**Skew**	Right Angles
Logged by	Davis/Stevens/Dietiker				**Operator**	Lamberson/Wilde/Dodds
Equipment	Failing 1500				**Drillers Hole No.**	A-96-53
Hole Stab. by	Casing				**Date of Work**	10/23/96

Bent	**Boring**	**Station**	**Location**	**Barge Deck Elevation**
22	B-14	0+117.7	C/L	209.16
Formerly	B-14	11+249	C/L	209.16

SOIL CLASSIFICATION TEST DATA					UNCONFINED COMPRESSIVE DATA		
Depth, m	**LL**	**PI**	**ASTM Class**		**Depth, m**	**Qu, kPa**	**P.P., kg/cm²**
9.50		NP	SP		18.90	1907	9.0+
					20.30	914	9.0+
					24.20	3412	9.0+
					26.60	7806	9.0+
					28.60	2662	9.0+
					29.30	1948	9.0+
					32.10	2855	9.0+
					33.10	423	9.0+

SIEVE ANALYSIS (Percent Passing)
AASHTO T88

Depth, m

	9.50							
19mm	95.00							
9.5mm	95.00							
4.75mm	95.00							
2.00mm	95.00							
.850mm	94.00							
.425mm	68.00							
.300mm	26.00							
.150mm	6.00							
.075mm	3.00							

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Boring B-14   0+117.7 C/L
Bent 22   11/04/96

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Job No.	J4P1102					Design	A5664
County	Ray/Lafayette		Route	13		Skew	Right Angles
Over	Missouri River					Operator	Wilde
Logged by	Miller					Drillers Hole No.	H-98-62
Equipment	Failing 1500					Date of Work	09/09/98, 09/15/98, 09/16/98
Hole Stab. by	Casing					Drill No.	G-7888
Automatic Hammer Efficiency					73 %		

Bent	Station	Location	Surface Elevation	LOG OF MATERIALS *	
Test Shaft #2	0+167.25	29.4m LT.	210.28	0.0-5.0m	Brown lean clay to silt.
CORING LOG (NX Double Tube Barrel)				5.0-6.20m	Gray silty clay, scattered gravel, medium stiff.

From	To	Run	Rec	Loss	% RQD	Notes	LOG OF MATERIALS (cont.)	
							6.20-12.47m	Gray fine sand, dense.
12.80	14.15	1.35	0.70	0.65			12.47-12.95m	Granite boulder.
14.85	16.35	1.50	1.46	0.04			12.95-14.50m	Weathered shaley limestone or boulders, lost water.
16.35	17.85	1.50	1.50	0.00			14.50-15.15m	Gray clay shale.
17.85	19.35	1.50	1.50	0.00			15.15-16.77m	Gray, fine grained sandstone, moderately hard.
19.35	20.85	1.50	1.50	0.00			16.77-17.57m	Gray, calcareous, micaceous silt shale, medium hard.
20.85	22.35	1.50	1.50	0.00			17.57-18.52m	Gray fine grained sandstone, moderately hard.
22.35	23.70	1.35	1.35	0.00			18.52-22.65m	Dark gray clay shale, moderately hard.
23.70	25.20	1.50	1.50	0.00			22.65-23.02m	Black shale, hard.
25.20	26.70	1.50	1.50	0.00			23.02-23.32m	Coal.
26.70	28.20	1.50	1.50	0.00			23.32-30.15m	Gray, calcareous, micaceous silt shale, moderately hard.
28.20	29.70	1.50	1.50	0.00			30.15-32.84m	Dark gray, thinly laminated clay shale, moderately hard.
29.70	31.20	1.50	1.50	0.00			32.84-33.85m	Black shale, hard.
31.20	32.70	1.50	1.50	0.00			33.85-34.20m	Coal.
32.70	34.20	1.50	1.50	0.00			34.20-36.08m	Gray clay shale, poorly laminated, soft.
34.20	35.70	1.50	1.50	0.00			36.08-36.47m	Gray shaley limestone, thick bedded, hard.
35.70	37.20	1.50	1.50	0.00			36.47-37.51m	Black clay shale, limestone seam at 36.9m,
37.20	38.70	1.50	1.50	0.00				moderately hard.
38.70	40.20	1.50	1.50	0.00			37.51-37.96m	Coal.
40.20	41.70	1.50	1.50	0.00			37.96-40.72m	Gray clay shale, poorly laminated, soft to medium hard,
41.70	43.15	1.45	1.45	0.00				limestone seam at 39.95m.

UNCONFINED COMPRESSIVE STRENGTH TEST DATA				40.72-41.00m	Gray shaley limestone, hard.
Depth, m	Qu, kPa	P.P., kg/cm²		41.00-43.15m	Gray clay shale, poorly laminated, soft to medium hard.
26.45	2140	9.0+			
27.20	1020	9.0+			
28.40	4500	9.0+			
29.60	3650	9.0+			
31.10	2330	9.0+			
33.70	31190	9.0+			
35.20	2290	9.0+			
36.0	310	9.0+			
38.40	600	9.0+			
39.60	340	9.0+			
41.60	150	7.0			
42.30	620	9.0+			
43.10	195	9.0+			

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MISSOURI DEPARTMENT OF TRANSPORTATION
Division of Materials

BORING DATA (CORE & SPT)

Job No.	J4P1102						
County	Lafayette/Ray		Route	13		Design	A5664
Over	Missouri River					Skew	Right Angles
Logged by	Fennessey/Stevens					Operator	Dodds
Equipment	CME 850					Drillers Hole No.	V-98-11
Hole Stab. by	Water					Date of Work	06/03 & 04/98
Automatic Hammer Efficiency				81	%	Drill No.	G-7950

Bent	Station	Location	Surface Elevation		LOG OF MATERIALS *
23	0+175.7	9m LT.	210.18	0.0-1.22m	Brown lean clay to silt, soft.
Boring F-39				1.22-4.72m	Light brown silty fine grained sand, very loose.

TEST DATA					4.72-10.36m	Gray-brown silt, soft, interlayered scattered fine to

Depth, m	SPT Blows/15cm	N_{60}	Pocket Pen., kg/cm²			medium grained sand.
1.50	2-2-2	5	Sand		10.36-11.28m	Gray, medium to coarse grained sand, trace brown
4.50	2-1-2	4	0.75			lean clay seams, loose, moist.
7.50	4-4-5	12	Sand		11.88-12.28m	Weathered limestone.
10.50	2-3-3	8	Sand		12.28-12.58m	Gray, fine to medium grained limestone, fractured,

UNCONFINED COMPRESSIVE STRENGTH						hard.
TEST DATA					12.58-13.50m	Purple clay shale, soft to medium hard.
					13.50-14.69m	Gray, poorly laminated clay shale, medium hard.

Depth, m	Qu, kPa	P.P., kg/cm²		14.69-15.75m	Gray, calcareous, micaceous silt shale, medium hard.
14.60	140	2.75		15.75-16.85m	Gray, fine grained sandstone, moderately hard.
15.80	12,255	9.0+		16.85-17.60m	Gray, calcareous, micaceous silt shale, medium hard.
18.30	7520	9.0+		17.60-18.40m	Gray, fine grained sandstone, moderately hard.
18.90	2060	9.0+		18.40-18.56m	Gray, calcareous, micaceous silt shale, hard.
20.40	2440	9.0+		18.56-20.97m	Gray, slightly calcareous clay shale, hard.
21.90	1120	9.0+		20.97-22.89m	Dark gray shale, poorly laminated, hard.
23.50	5465	9.0+		22.89-23.11m	Black shale, hard.

CORING LOG (NX Double Tube Barrel)							23.11-23.24m	Black coal, hard but brittle.
From	To	Run	Rec	Loss	% RQD	Notes	23.24-24.26m	Gray and black banded, slightly calcareous, micaceous
12.28	13.81	1.53	0.61	0.91	0**			silt shale, medium hard.
13.81	15.33	1.52	1.52	0.00				
15.33	16.85	1.52	1.52	0.00				
16.85	18.40	1.55	1.55	0.00				
18.40	19.90	1.50	1.50	0.00				
19.90	21.21	1.31	1.31	0.00				
21.21	22.74	1.53	1.37	0.16				
22.74	24.26	1.52	1.52	0.00				

WATER TABLE OBSERVATIONS						
Date	Time Change	Depth Hole Open		Depth To Water		
06/09/98	5 days	5.30m		1.0m	**RQD on limestone portion only.	

N_{60} - Corrected N value for standard 60% SPT efficiency.

$N_{60} = (Em/60)Nm$ Em - Measured transfer efficiency in percent.

Nm - Observed N-value.

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Boring F-39   0+175.7, 9m LT.
Bent 23

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APPENDIX G
CONSTRUCTION AND LOAD TEST PHOTOGRAPHS FOR GRANDVIEW
TEST SITE (HNTB 2002)

G1- View of test site from North

G2- 2550 mm (72 in) bullet tooth rock auger

G3- Installing "knuckle in 2550 mm (72 in) core barrel

G4- 914 mm (36in) Core barrel

G5- Clean Out bucket

G6- Large piece of Westerville Limestone core

G7- Sonic Caliper

G8- Load frame arrives at test site

G9- Down Hole video camera

G10- Video camera monitor and controls

G11- LVWDT installed above compression device

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G12- Installed sister bar Strain Gage

G13- Raising Load frame

G14- Installation of rollers/spacers along load frame

G15- Scoring CSL pipe

G16- Scored CSL pipe

G17- Lowering frame into rock socket

G18- Welding load frame to supports on temporary casing

G19- Pump truck and tremie

G20- Lowering 127mm (5 in) tremie pipe

G21- View of load test in progress

G22- Reference beam and load test in progress

G23- Top of shaft instrumented for load test

APPENDIX H
BORING LOGS AND CORE PHOTOGRAPHS FOR GRANDVIEW TEST SITE

MISSOURI DEPARTMENT OF TRANSPORTATION
Project Operations

BORING DATA (CORE & SPT)

Job No.:	J4I0766E				**Design:**	A6251	
County:	Jackson	**Route:**	I-470				
Over:	Hickman Mills Dr., Ramp S-W, Ramp S-W Detour (B), & Hickman Mills Creek (S)				**Skew:**	Right Angles	
Logged by:	Davis				**Operator:**	Murray	
Equipment:	Versa Drill 4000 TR-2, Split Spoon Sampler, NX Core Barrel				**Drillers Hole No.:**	Y-02-34	
Hole Stab. by:	Hollow Stem Augers				**Date of Work:**	04/03/02	
Automatic Hammer Efficiency:			73 %		**Drill No.:**	G-8641	

Bent	Station	Location	Surface Elevation		LOG OF MATERIALS*
12	25+37	0.7' RT.	938.5	0.0-2.1'	Reddish-brown shaley lean to fat clay,
BH9-12-2					moist, medium stiff.

TEST DATA				
Depth, ft.	**SPT Blows/6"**	**N_{60}**	**Pocket Pen., tsf**	**Est. Equiv., Qu, tsf**
5.0	1-5-77 in 2"	100	2.0	4.3
11.1	4-5-9	17	3.2	

LOG OF MATERIALS* (continued)

Depth	Description
2.1-6.1'	Gray shaley fat clay to clay shale, moist to 5.3', then dry, stiff if clay, soft if shale.
6.1-6.5'	Gray, fine grained, thin bedded limestone, moderately hard, probably a boulder.
6.5-15.0'	Gray and brown shaley fat clay, moist, stiff to very stiff, tried to core 6.1 to 11.1' with near zero recovery, back on solid shale at 14.6'.
15.0-21.0'	Gray thinly laminated clay shale, soft.
21.0-26.6'	Gray, thin bedded, fine grained limestone.
26.6-32.6'	Dark gray thinly laminated clay shale, very soft to soft.
32.6-33.7'	Gray, thin bedded, fine grained limestone, medium hard.
33.7-33.9'	Dark gray thinly laminated clay shale, soft.
33.9-40.4'	Gray, thin bedded, fine grained limestone, medium to moderately hard, unweathered.
40.4-42.2'	Dark gray thinly laminated clay shale, soft.

UNCONFINED COMPRESSIVE STRENGTH TEST DATA

Depth, ft.	Qu, tsf	P.P., tsf
37.6	884.2	>9.0

CORING LOG (NX Double Tube Barrel)

From	To	Run	Rec	Loss	% RQD	Notes
6.1	11.1	5.0	0.8	4.2	0	
15.0	20.0	5.0	2.5	2.5	0	
20.0	25.0	5.0	5.0	0	8**	
25.0	29.2	4.2	4.2	0	18**	
29.2	32.2	3.0	2.9	0.1	30	
32.2	37.2	5.0	4.6	0.4	36	
37.2	42.2	5.0	5.0	0	26	

**RQD on limestone portion only.

WATER TABLE OBSERVATIONS

Date	Time Change	Depth Hole Open	Depth To Water

N_{60} - Corrected N value for standard 60% SPT efficiency.

N_{60} = (Em/60)Nm   Em - Measured transfer efficiency in percent.

Nm - Observed N-value.

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MISSOURI DEPARTMENT OF TRANSPORTATION
Project Operations

BORING DATA (CORE & SPT)

	Sheet	16	of	17

Job No.:	J4I0766E				**Design:**	A6251
County:	Jackson	**Route:**	I-470		**Skew:**	Right Angles
Over:	Hickman Mills Dr., Ramp S-W, Ramp S-W Detour (B), & Hickman Mills Creek (S)					
Logged by:	Davis				**Operator:**	Murray
Equipment:	Versa Drill 4000 TR-2, Split Spoon Sampler, NX Core Barrel				**Drillers Hole No.:**	Y-02-32
Hole Stab. by:	Hollow Stem Augers				**Date of Work:**	04/02/02
Automatic Hammer Efficiency:			73 %		**Drill No.:**	G-8641

Bent	Station	Location	Surface Elevation		LOG OF MATERIALS*
13	26+19	31.3' LT.	950.5	0.0-1.9'	Reddish-brown lean to fat clay, moist,
BH9-13-2					medium stiff.

TEST DATA

Depth, ft.	SPT Blows/6"	N_{60}	Pocket Pen., tsf		Log continued
				1.9-3.3'	Gray lean clay, moist, medium stiff.
5.0	4-5-9	17	2.35	3.3-6.7'	Light brown lean clay, trace gravel,
10.0	5-3-3	7	4.20		medium stiff.
15.0	12-18-36	66	>9.0	6.7-7.9'	Gray silt shale, very soft, dry.
20.0	22-30-41	86	6.1	7.9-12.9'	Gray calcareous silt shale, with limestone
					pockets, dry, medium hard.
				12.9-21.8'	Gray silt shale, soft, yellow-brown mottles,
					thinly laminated, cut with hollow stem
					augers, split spoon, and rockbit.
				21.8-27.1'	Gray thinly laminated silt and clay shale,
					soft.
				27.1-32.9'	Reddish-brown thinly laminated clay shale,
					very soft to soft.
				32.9-34.3'	Gray thinly laminated silt shale, soft,
					unweathered.
				34.3-36.0'	Gray, thin to medium bedded, fine grained
					limestone, moderately hard, unweathered.
				36.0-36.8'	Gray thinly laminated clay shale, scattered
					limestone nodules, soft.
				36.8-39.6'	Gray, thin bedded, fine grained limestone,
					medium hard, slightly weathered.
				39.6-45.0'	Gray thinly laminated silt shale to clay
					shale or claystone, very soft.

CORING LOG (NX Double Tube Barrel)

From	To	Run	Rec	Loss	% RQD	Notes
21.8	26.8	5.0	5.0	0	0	
26.8	31.8	5.0	4.1	0.9	0	
31.8	36.8	5.0	5.0	0	20**	
36.8	41.8	5.0	5.0	0	0**	
41.8	45.0	3.2	3.2	0	0	

**RQD on limestone portion only.

UNCONFINED COMPRESSIVE STRENGTH TEST DATA

Depth, ft.	Qu, tsf	P.P., tsf
29.2	2.7	4.25
35.3	458.3	>9.0
38.2	2.4	3.20

WATER TABLE OBSERVATIONS

Date	Time Change	Depth Hole Open	Depth To Water

N_{60} - Corrected N value for standard 60% SPT efficiency.

$N_{60} = (Em/60)Nm$ Em - Measured transfer efficiency in percent.

Nm - Observed N-value.

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MISSOURI DEPARTMENT OF TRANSPORTATION
Project Operations - Materials

BORING DATA (CORE & SPT)

	Sheet	5	of	14

Job No.:	J4I0766E							
County:	Jackson		Route:	I-470		Design:	A6252	
Over:	US 71					Skew:	Right Angles	
Logged by:	Hilchen					Operator:	Wineland	
Equipment:	Failing 1500					Drillers Hole No.:	A-02-48	
Hole Stab. by:	Drilling Fluids/Casing					Date of Work:	05/02/02	
Automatic Hammer Efficiency:				72	%	Drill No.:	G-7887	

Bent	Station	Location	Surface Elevation	LOG OF MATERIALS*	
(BH8-4-2) 4	44+71	50.8' RT.	934.6	0.0-1.8'	Brown lean clay, with gravel, stiff, moist.
				1.8-4.9'	Gray shale fill, very stiff, moist.
		TEST DATA		4.9-8.8'	Brown fat clay, stiff, moist.
Depth, ft.	**SPT Blows/6"**	**N₆₀**	**Pocket Pen., tsf**	8.8-13.9'	Light gray, fine grained, medium bedded,
					medium hard limestone, weathered clay filled
					seams from 8.8 to 11.1'.
				13.9-15.8'	Olive-gray to gray shale, thickly laminated,
					very soft rock.
				15.8-16.8'	Dark gray thinly laminated shale, very soft
					rock.
				16.8-19.1'	Bluish-gray thickly laminated shale,
					very soft rock.
				19.1-27.7'	Light gray, fine grained, thin to medium
					bedded, soft to moderately hard limestone,
					with thin bluish-gray shale lenses.
				27.7-54.5'	Dark gray thinly laminated shale,
					very soft rock.

Note: In the column headers above, N_{60} denotes the corrected N value for standard 60% SPT efficiency.

UNCONFINED COMPRESSIVE STRENGTH
TEST DATA

Depth, ft.	Qu, tsf	P.P., tsf
12.6	410.2	>9.0
19.7	137.9	>9.0
23.9	707.3	>9.0
28.1	8.4	>9.0
36.0	29.9	>9.0
42.0	25.6	>9.0
48.2	24.9	>9.0
52.1	30.5	>9.0

CORING LOG (NX Double Tube Barrel)

From	To	Run	Rec	Loss	% RQD	Notes
4.5	8.5	4.0	1.3	2.7	0	
8.5	13.5	5.0	3.8	1.2	30	
13.5	18.5	5.0	4.7	0.3	100**	
18.5	23.5	5.0	4.7	0.3	39**	
23.5	26.5	3.0	3.0	0	70	
26.5	31.5	5.0	5.0	0	Shale	
31.5	34.5	3.0	3.0	0	Shale	
34.5	39.5	5.0	5.0	0	Shale	
39.5	44.5	5.0	5.0	0	Shale	
44.5	49.5	5.0	5.0	0	Shale	
49.5	54.5	5.0	5.0	0	Shale	

WATER TABLE OBSERVATIONS

Date	Time Change	Depth Hole Open	Depth To Water

N_{60} - Corrected N value for standard 60% SPT efficiency.

$N_{60} = (Em/60)Nm$ Em - Measured transfer efficiency in percent.

Nm - Observed N-value.

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MISSOURI DEPARTMENT OF TRANSPORTATION
Project Operations - Materials

BORING DATA (CORE & SPT)

Sheet	6	of	14

Job No.:	J4I0766E						
County:	Jackson		Route:	I-470		Design:	A6252
Over:	US 71					Skew:	Right Angles
Logged by:	Inglish					Operator:	Hees
Equipment:	Versa Drill 4000 TR-2					Drillers Hole No.:	B-02-27
Hole Stab. by:	Hollow Stem Augers					Date of Work:	04/24/02
Automatic Hammer Efficiency:				76	%	Drill No.:	G-8690

Bent	Station	Location	Surface Elevation		LOG OF MATERIALS*
(BH8-5-2) 5	46+22	68.8' RT.	935.2	0.0-5.8'	Brown fat clay, very soft, moist.
				5.8-7.6'	Grayish-brown shaley fat clay, medium stiff, moist.

TEST DATA

Depth, ft.	SPT Blows/6"	N_{60}	Pocket Pen., tsf		
5.0	2-3-4	9	1.0	7.6-10.6'	Red shaley clay, stiff, moist.
10.0	4-7-6	16	1.5	10.6-19.3'	Light brown shaley fat clay, scattered gravel,
15.0	4-10-14	30	2.0		stiff, moist.
				19.3-25.1'	Light gray medium bedded limestone,
					fine grained, soft.
				25.1-29.8'	Dark gray shale, soft rock.

SOIL CLASSIFICATION TEST DATA

Depth, ft.	LL	PI	ASTM Class		
5.0	50	28	CH	29.8-33.2'	Light gray shale, very soft rock.
				33.2-37.6'	Light gray limestone, medium to coarse grained,
					medium bedded, soft to moderately hard rock,
					good quality.
				37.6-47.4'	Dark gray shale, very soft rock.

UNIT WEIGHTS

Depth, ft.	γsat, pcf	γmoist, pcf	sat%
5.0	120.8		100[1]
15.0	136.2		100[1]

[1] Assumed
[2] Actual

CORING LOG (NX Double Tube Barrel)

From	To	Run	Rec	Loss	% RQD	Notes
19.3	24.3	5.0	5.0	0	30	
24.3	29.3	5.0	5.0	0	67**	
29.3	33.6	4.3	4.1	0.2	Shale	
33.6	38.6	5.0	4.8	0.2	90	
38.6	43.6	5.0	5.0	0	Shale	
43.6	47.4	3.8	3.8	0	Shale	

UNCONFINED COMPRESSIVE STRENGTH
TEST DATA

Depth, ft.	Qu, tsf	P.P., tsf
20.2	240.7	>9.0
35.4	861.0	>9.0
36.6	17.5	>9.0
44.2	32.6	>9.0

**RQD on limestone portion only.

WATER TABLE OBSERVATIONS

Date	Time Change	Depth Hole Open	Depth To Water
04/25/02	22 hours	45.4'	6.0'

N_{60} - Corrected N value for standard 60% SPT efficiency.

$N_{60} = (Em/60)Nm$ Em - Measured transfer efficiency in percent.

Nm - Observed N-value.

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Rev. 05/01

MISSOURI DEPARTMENT OF TRANSPORTATION
Project Operations - Materials

BORING DATA (CORE & SPT)

	Sheet	7	of	14

Job No.:	J4I0766E						
County:	Jackson		Route:	I-470		Design:	A6252
Over:	US 71					Skew:	Right Angles
Logged by:	Hilchen					Operator:	Dodds
Equipment:	CME 45					Drillers Hole No.:	T-02-32
Hole Stab. by:	Hollow Stem Augers					Date of Work:	05/20/02
Automatic Hammer Efficiency:				83	%	Drill No.:	G-7965

Bent	Station	Location	Surface Elevation	LOG OF MATERIALS*	
(BH8-6-2) 6	47+73	51.4' RT.	938.0	Inaccessible due to safety barrier curb and underground utilities.	
Offset to:	47+77.5	58.1' RT.	938.0	0.0-0.3'	Asphalt.
TEST DATA				0.3-1.1'	Concrete.
Depth, ft.	**SPT Blows/6"**	**N$_{60}$**	**Pocket Pen., tsf**	1.1-1.6'	Gravel road base.
				1.6-10.6'	Gray shale, very soft rock, scattered calcareous
					seams throughout.
				10.6-22.7'	Reddish-brown, thickly laminated, extremely
					soft shale, slightly weathered.
				22.7-28.4'	Light gray, fine grained, medium bedded,
					medium hard limestone.
				28.4-35.0'	Greenish-gray to gray thinly to thickly
					laminated shale, very soft rock.
				35.0-41.0'	Light gray, fine grained, medium bedded,
					medium hard limestone, highly weathered zone
					from 35.9 to 36.8'.
				41.0-48.4'	Greenish-gray to dark gray shale, thinly
					laminated, very soft rock.

UNCONFINED COMPRESSIVE STRENGTH
TEST DATA

Depth, ft.	Qu, tsf	P.P., tsf
31.1	16.0	>9.0
42.0	20.0	>9.0
46.6	33.1	>9.0

CORING LOG (NX Double Tube Barrel)

From	To	Run	Rec	Loss	% RQD	Notes
5.7	10.7	5.0	5.0	0	Shale	
10.7	15.0	4.3	4.3	0	Shale	
15.0	20.0	5.0	4.4	0.6	Shale	
20.0	25.0	5.0	5.0	0	78**	
25.0	30.0	5.0	5.0	0	88	
30.0	35.0	5.0	5.0	0	Shale	
35.0	39.3	4.3	3.4	0.9	40	
39.3	43.4	4.1	4.1	0	65**	
43.4	48.4	5.0	4.6	0.4	Shale	

**RQD on limestone portion only.

WATER TABLE OBSERVATIONS

Date	Time Change	Depth Hole Open	Depth To Water

N$_{60}$ - Corrected N value for standard 60% SPT efficiency.

N$_{60}$ = (Em/60)Nm   Em - Measured transfer efficiency in percent.

Nm - Observed N-value.

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MISSOURI DEPARTMENT OF TRANSPORTATION
Project Operations - Materials

BORING DATA (CORE & SPT)

				Sheet	9	of	14

Job No.:	J4I0766E					
County:	Jackson	**Route:**	I-470	**Design:**	A6252	
Over:	EBL I-470			**Skew:**	Right Angles	
Logged by:	Hilchen			**Operator:**	Wineland	
Equipment:	Failing 1500			**Drillers Hole No.:**	A-02-45	
Hole Stab. by:	Drilling Fluids/Casing			**Date of Work:**	04/29/02	
Automatic Hammer Efficiency:			72 %	**Drill No.:**	G-7887	

Bent	Station	Location	Surface Elevation	LOG OF MATERIALS*
(BH8-7-2) 7	49+00	18.7' RT.	941.9	Inaccessible due to
Offset to:	49+07.7	31.6' RT.	942.3	0.0-4.8' Brown lean clay, with cobbles, rock fill.

TEST DATA

Depth, ft.	SPT Blows/6"	N_{60}	Pocket Pen., tsf		LOG OF MATERIALS
5.0	13-19-24	52	8.5	4.8-11.0'	Olive-brown weathered shale, extremely to very soft rock.
				11.0-15.8'	Dark gray shale, with gray micaceous banding, fine grained, laminated to thin bedded, very soft rock.
				15.8-17.2'	Gray to light gray calcareous layered shale to argillaceous limestone, fine grained, thin bedded, very soft to soft rock.
				17.2-21.3'	Bluish-gray blocky shale, very soft rock.
				21.3-23.8'	Reddish-brown thinly laminated shale, very soft rock.
				23.8-25.6'	Dark gray thinly laminated shale, very soft rock.
				25.6-30.3'	Light gray, fine grained, medium bedded, medium hard limestone.

UNCONFINED COMPRESSIVE STRENGTH TEST DATA

Depth, ft.	Qu, tsf	P.P., tsf
14.7	9.7	>9.0
22.8	10.1	>9.0

CORING LOG (NX Double Tube Barrel)

From	To	Run	Rec	Loss	% RQD	Notes
9.0	14.0	5.0	5.0	0	Shale	
14.0	19.0	5.0	5.0	0	Shale	
19.0	24.0	5.0	4.2	0.8	Shale	
24.0	29.0	5.0	5.0	0	56**	
29.0	30.3	1.3	1.3	0	54	

UNIT WEIGHTS

Depth, ft.	γsat, pcf	γmoist, pcf	sat%
5.0	136.0		100[1]

[1] Assumed
[2] Actual

****RQD on limestone portion only.**

WATER TABLE OBSERVATIONS

Date	Time Change	Depth Hole Open	Depth To Water

N_{60} - Corrected N value for standard 60% SPT efficiency.

$N_{60} = (Em/60)Nm$ Em - Measured transfer efficiency in percent.

Nm - Observed N-value.

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MISSOURI DEPARTMENT OF TRANSPORTATION
Project Operations - Materials

BORING DATA (CORE & SPT)

	Sheet	10	of	14

Job No.:	J4I0766E					
County:	Jackson	**Route:**	I-470	**Design:**	A6252	
Over:	US 71, Ramp S-W, Ramp S-W Detour & Hickman Mills Drive and Creek(S)			**Skew:**	Right Angles	
Logged by:	Davis			**Operator:**	Hees	
Equipment:	Versa Drill 4000 TR-2, Split Spoon Sampler, NX Core Barrel			**Drillers Hole No.:**	Y-02-39	
Hole Stab. by:	Hollow Stem Augers			**Date of Work:**	04/16/02	
Automatic Hammer Efficiency:			73 %	**Drill No.:**	G-8641	

Bent	Station	Location	Surface Elevation		LOG OF MATERIALS*
(BH8-8-1) 8	50+34	18.6' LT.	933.7	0.0-5.1'	Brown shaley fat clay, gray and yellowish-brown mottles, moist, very stiff.
				5.1-11.2'	Gray and brown shaley fat clay, scattered gravel, moist, very stiff to hard.

TEST DATA

Depth, ft.	SPT Blows/6"	N_{60}	Pocket Pen., tsf		
4.0	2-2-2	5	2.15	11.2-15.4'	Gray thinly laminated clay shale, soft, weathered.
9.0	1-2-3	6	5.5	15.4-19.2'	Gray, thin to medium bedded, fine grained limestone, slightly weathered, moderately hard.
14.0	4-5-10 in 2", then 10 blows, no advance	--	4.9	19.2-26.8'	Dark gray thinly laminated clay shale, very soft.
				26.8-34.0'	Gray, thin bedded, medium to coarse grained limestone, weathered, soft to moderately hard.
				34.0-45.4'	Dark gray thinly laminated clay shale, very soft.

UNCONFINED COMPRESSIVE STRENGTH
TEST DATA

Depth, ft.	Qu, tsf	P.P., tsf
17.6	571.4	>9.0
23.6	11.1	5.7
32.6	725.8	>9.0
40.8	19.4	>9.0

CORING LOG (NX Double Tube Barrel)

From	To	Run	Rec	Loss	% RQD	Notes
15.4	20.4	5.0	3.8	1.2	42	
20.4	25.4	5.0	5.0	0	0	
25.4	30.4	5.0	5.0	0	10**	
30.4	35.4	5.0	3.6	1.4	40	
35.4	40.4	5.0	5.0	0	0	
40.4	45.4	5.0	4.6	0.4	0	

**RQD on limestone portion only.

WATER TABLE OBSERVATIONS

Date	Time Change	Depth Hole Open	Depth To Water	

N_{60} - Corrected N value for standard 60% SPT efficiency.

$N_{60} = (Em/60)Nm$ Em - Measured transfer efficiency in percent.

Nm - Observed N-value.

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MISSOURI DEPARTMENT OF TRANSPORTATION
Project Operations

BORING DATA (CORE & SPT)

	Sheet	1	of	2

Job No.:	J4I0766D					
County:	Jackson		**Route:**	I-470	**Design:**	A6252
Over:	Route 71				**Skew:**	R.A. to B/L Rte. 71
Logged by:	Davis				**Operator:**	Hees
Equipment:	Failing 1500, Split Spoon Sampler, NX Wireline Core Barrel				**Drillers Hole No.:**	A-103-01
Hole Stab. by:	Casing				**Date of Work:**	12/17/01, 12/20/01
Automatic Hammer Efficiency:				72 %	**Drill No.:**	G-7887

Bent	Station	Location	Surface Elevation		LOG OF MATERIALS*
	125+05	90' LT.	942.2	0.0-0.5'	Brown lean clay, soft, moist.
				0.5-3.1'	Rock, medium hard, probably weathered limestone.

TEST DATA

Depth, ft.	SPT Blows/6"	N_{60}	Pocket Pen., tsf	Est. Equiv., Qu, tsf		
20.0	40-43 in 3"	100	6.1	4.0	3.1-3.9'	Clay seam, soft.
					3.9-4.7'	Rock, soft, probably weathered shale.
					4.7-6.5'	Brown clay, soft, wet.

UNCONFINED COMPRESSIVE STRENGTH TEST DATA

Depth, ft.	Qu, tsf	Pocket Pen., tsf	Wn%		
				6.5-15.5'	Gray shaley clay, soft, wet.
22.2	5.18	6.5	14.8	15.5-26.6'	Gray clay shale, with brown clay shale
23.9	4.61	7.2	9.9		seams, soft, thinly laminated to
27.4	118.6	>9.0	--		thickly laminated, very poor quality.
32.2	14.26	>9.0	9.3	26.6-32.0'	Gray, fine grained, medium to thick bedded
36.1	2.88	5.5	12.4		limestone, weathered, medium hard.
39.1	164.8	>9.0	--	32.0-33.7'	Greenish-gray thinly laminated silt shale,
43.2	411.6	>9.0	--		medium hard.
46.2	13.8	>9.0	10.1	33.7-34.4'	Black thinly laminated silt shale, soft.
49.5	32.7	>9.0	8.8	34.4-37.7'	Gray thinly laminated silt shale, soft.
51.4	18.9	>9.0	8.6	37.7-44.3'	Gray and yellowish-brown, medium
54.5	27.6	>9.0	8.8		bedded, coarse grained, oolitic limestone,
61.1	20.7	>9.0	8.6		weathered, medium hard, lost circulating
67.9	25.9	>9.0	8.6		water at 40.1'.
				44.3-77.2'	Greenish-gray and gray thinly laminated silt
					and clay shale, soft to medium hard.
				77.2-78.2'	Gray, thickly laminated to thin bedded,
					medium to coarse grained limestone, with
					shale lamina, medium hard, unweathered.

CORING LOG (NX Double Tube Barrel)

From	To	Run	Rec	Loss	% RQD	Notes
21.0	26.0	5.0	5.0	0	0	
26.0	31.0	5.0	5.0	0	78**	
31.0	36.0	5.0	5.0	0	14**	
36.0	41.0	5.0	5.0	0	58**	
41.0	46.0	5.0	4.1	0.9	30**	
46.0	51.0	5.0	4.2	0.8	0	
51.0	56.0	5.0	5.0	0	0	
56.0	61.0	5.0	2.1	2.9	0	
61.0	66.0	5.0	3.7	1.3	0	
66.0	71.0	5.0	3.9	1.1	0	
71.0	76.0	5.0	3.1	1.9	0	
76.0	78.2	2.2	1.7	0.5	0	

**RQD on limestone portion only.

WATER TABLE OBSERVATIONS

Date	Time Change	Depth Hole Open	Depth To Water

UNIT WEIGHTS

Depth, ft.	γmoist, pcf	γsat, pcf	%sat
22.2		139.0	100[2]
23.9		146.1	100[2]
27.4		166.3	N/A
32.2		147.0	100[2]
36.1	138.2		93.1[2]
39.1		162.5	N/A
43.2		162.0	N/A
46.2		146.8	100[2]
49.5		150.1	100[2]
51.4	142.6		85.4[2]
54.5		152.0	100[2]
61.1		151.6	100[2]
67.9	146.0		95.7[2]

[1] Assumed
[2] Actual

N_{60} - Corrected N value for standard 60% SPT efficiency.

$N_{60} = (Em/60)Nm$ Em - Measured transfer efficiency in percent.

Nm - Observed N-value.

MISSOURI DEPARTMENT OF TRANSPORTATION
Project Operations

BORING DATA (CORE & SPT)

	Sheet	2	of	2

Job No.:	J4I0766D					**Design:**	A6252
County:	Jackson		**Route:**	71		**Skew:**	R.A.
Over:	Exploratory Bore for Osterberg Test Shaft					**Skew:**	R.A.
Logged by:	Davis					**Operator:**	Hees
Equipment:	Failing 1500, Split Spoon Sampler					**Drillers Hole No.:**	A-01-104
Hole Stab. by:	Drilling Fluids					**Date of Work:**	12/21/01
Automatic Hammer Efficiency:				72 %		**Drill No.:**	G-7887

Bent	Station	Location	Surface Elevation		LOG OF MATERIALS*
	125+05	80' LT.	942.1	0.0-0.6'	Brown lean clay, moist, medium stiff.
				0.6-3.5'	Rock, medium hard, probably limestone,

TEST DATA						cut with rockbit.
Depth, ft.	SPT Blows/6"	N$_{60}$	Pocket Pen., tsf	Est. Equiv., Qu, tsf	3.5-3.9'	Brown clay shale, soft.
5.0	15-30-53 in 4½"	100	5.1	3.8	3.9-4.8'	Rock, medium hard, probably limestone,
10.0	65-18 in 1"	100	4.2	4.4		cut with rockbit.
15.0	83 in 3"	100	>9.0	9.7	4.8-15.2'	Gray calcareous silt shale, soft, thinly
						laminated.

CORING LOG (NX Double Tube Barrel)

From	To	Run	Rec	Loss	% RQD	Notes

WATER TABLE OBSERVATIONS

Date	Time Change	Depth Hole Open	Depth To Water

N$_{60}$ - Corrected N value for standard 60% SPT efficiency.

$N_{60} = (Em/60)Nm$ Em - Measured transfer efficiency in percent.

Nm - Observed N-value.

* Persons using this information are cautioned that the materials shown are determined by the equipment noted and accuracy of the "log of materials" is limited thereby and by judgment of the operator. THIS INFORMATION IS FOR DESIGN PURPOSES ONLY.

APPENDIX I
**CONSTRUCTION AND LOAD TEST PHOTOGRAPHS FOR WAVERLY TEST
SITE**

I1- Pier 11 at Waverly site.

I2- Pier 11 at Waverly site.

I3- American 9270 Series crane with a Hain twin drill, drilling rock socket at Pier 12, existing bridge in background.

I4- Temporary outer casing, inner permanent casing, and casing clamp at Pier 12 (Pier 11 is in the background).

I-5- Pier 10 on North river bank

I6- Bullet tooth rock auger used to excavate rock socket at Waverly test site.

I7- Core Barrel used to excavate rock socket at Waverly test site.

I8- Vibratory Hammer

I9- Using vibratory hammer to set casing at Pier 12.

I10- Miniature shaft inspection device (Mini-SID) used to inspect bottom of rock sockets at Waverly bridge site.

I11- Mini-SID inspecting shaft.

I12- Rebar cage with Osterberg load cell.

I13- Portable Slurry Plant

I14- Delivering concrete to site

I-15- Using tremie to place concrete.

APPENDIX J
BORING LOGS AND CORE PHOTOGRAPHS FOR WAVERLY TEST SITE

MISSOURI DEPARTMENT OF TRANSPORTATION
Division of Materials
BORING DATA (CORE & SPT)

Job No.	J2P0639						
County	Carroll/Lafayette		**Route**	65.00	**Design**	A5910	
Over	Missouri River				**Skew**	Right Angles	
Logged by	Stevens				**Operator**	Wineland	
Equipment	Failing 1500				**Drillers Hole No.**	L-00-13	
Hole Stab. by	Casing				**Date of Work**	04/25/00	
Automatic Hammer Efficiency				73.00 %	**Drill No.**	G-7889	

Bent	Station	Location	Elevation		LOG OF MATERIALS *
11.00	95+63	28' LT.	--		Unable to position due to current.
Offset to:	95+60.4	30.1' LT.	663.20		0.0-6.1' Barge deck.
TEST DATA			657.10		6.1-27.1' Water.

Depth, ft.	SPT Blows/6"	N_{60}	$(N_1)_{60}$	Pocket Pen., tsf	Est. Equiv., Qu, tsf	Elevation		LOG OF MATERIALS *
						636.10		27.1-35.0' Brown and gray medium to coarse grained sand, scattered fine gravel, dense.
30.0	23-27-10	45	53	Sand		628.20		35.0-47.0' Gray coarse grained sand, scattered
40.0	6-6-6	15	15	Sand				fine gravel, medium dense.
50.0	13-13-11	29	26	Sand		616.20		47.0-52.8' Gray fine to medium grained silty sand, dense.
73.1	82 in 5.5"	100		9.00	5.5	610.40		52.8-58.1' Gray and purple claystone, soft.
98.7	82 in 2"	100.00		9.00+	14.4	605.10		58.1-63.1' Greenish-gray clay shale, soft.
						600.10		63.1-64.5' Dark gray clay shale.
						598.70		64.5-64.7' Coal.
						598.50		64.7-67.9' Gray claystone, poorly laminated, soft to medium hard.
						595.30		67.9-68.3' Coal.
						594.90		68.3-68.4' Dark brown claystone seam.
						594.80		68.4-71.5' Gray micaceous silt shale, moderately hard, slightly calcareous.
						591.70		71.5-78.5' Gray clay shale to claystone, soft.

CORING LOG (NX Double Tube Barrel)						Elevation		LOG OF MATERIALS *
From	To	Run	Rec	Loss	% RQD	584.70		78.5-85.6' Black carbonaceous shale, moderately hard, laminated.
53.10	58.10	5.0	4.80	0.20	Shale	577.60		85.6-90.2' Black coal, fossiliferous, brittle,
58.10	63.10	5.0	4.80	0.20	Shale			claystone seam at 88.4 to 88.5'.
63.10	68.10	5.0	4.30	0.70	Shale	573.0		90.2-99.0' Black carbonaceous clay shale,
68.10	73.10	5.0	5.0	0.00	Shale			moderately hard.
73.70	78.70	5.0	4.50	0.50	Shale	564.2		99.0-100.5' Brown and gray coarse grained fossiliferous
78.70	88.70	10.0	10.0	0.00	Shale			irregularly bedded limestone, medium bedded,
88.70	98.70	10.0	9.7	0.30	Shale			hard.
98.90	103.90	5.0	5.0	0.00	80.00	562.70		100.5-103.6' Brown and gray coarse grained sandstone,

WATER TABLE OBSERVATIONS							LOG OF MATERIALS *
Date		Time Change	Depth Hole Open	Depth To Water			with black and white laminations,
							medium bedded, hard.
					559.60		103.6-103.9' Brownish-gray claystone, hard.
					559.30		Boring terminated.

N_{60} = (Em/60)Nm N_{60} - Corrected N value for standard 60% SPT efficiency. Em - Measured transfer efficiency in percent.   Nm - Observed N-value.

$(N_1)_{60}$ = Normalized standardized blow count corrected for effective overburden pressure.

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MISSOURI DEPARTMENT OF TRANSPORTATION
Division of Materials
BORING DATA (CORE & SPT)

Job No.	J2P0639						
County	Carroll/Lafayette		**Route**	65.00		**Design**	A5910
Over	Missouri River					**Skew**	Right Angles
Logged by	Stevens					**Operator**	Wineland
Equipment	Failing 1500					**Drillers Hole No.**	L-00-13
Hole Stab. by	Casing					**Date of Work**	04/25/00
Automatic Hammer Efficiency				73.00	%	**Drill No.**	G-7889

Bent	**Station**	**Location**	**Elevation**		**LOG OF MATERIALS ***		
11.00	95+63	28' LT.	--				
Offset to:	95+60.4	30.1' LT.	663.20				

TEST DATA

Depth, ft.	**SPT Blows/6"**	**N_{60}**	**$(N_1)_{60}$**	**Pocket Pen., tsf**	**Est. Equiv., Qu, tsf**		

UNCONFINED COMPRESSIVE STRENGTH TEST DATA

Elev.	**Depth, ft.**	**Qu, tsf**	**P.P., tsf**
608.4	54.8	1.6	9.0+
601.8	61.4	5.3	9.0+
597.4	65.8	0.7	9.0+
594.0	69.2	9.1	9.0+
586.7	76.5	7.9	9.0+
578.0	85.2	37.2	9.0+
566.5	96.7	18.8	9.0+

CORING LOG (NX Double Tube Barrel)

From	**To**	**Run**	**Rec**	**Loss**	**% RQD**

WATER TABLE OBSERVATIONS

Date	**Time Change**	**Depth Hole Open**	**Depth To Water**

$N_{60} = (Em/60)Nm$ N_{60} - Corrected N value for standard 60% SPT efficiency. Em - Measured transfer efficiency in percent. Nm - Observed N-value.

$(N_1)_{60}$ = Normalized standardized blow count corrected for effective overburden pressure.

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L-00-13
Bent 11 95+60.4 30.1' Lt.
4/25/00

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MISSOURI DEPARTMENT OF TRANSPORTATION
Division of Materials

BORING DATA (CORE & SPT)

Sheet __22a__ of __35__

Job No.	J2P0639					
County	Carroll/Lafayette		**Route**	65.00	**Design**	A5910
Over	Missouri River				**Skew**	Right Angles
Logged by	Davis				**Operator**	Lamberson
Equipment	Failing 1500				**Drillers Hole No.**	L-00-14
Hole Stab. by	Casing				**Date of Work**	04/25/00
Automatic Hammer Efficiency				73.00 %	**Drill No.**	G-7889

Bent	Station	Location		Elevation	LOG OF MATERIALS *		
11.00	95+63	56' LT.					
Offset to:	95+60.8	56.6' LT.		663.30	0.0-6.1'	Barge deck.	
TEST DATA				657.20	6.1-25.6'	Water.	
Depth, ft.	**SPT Blows/6"**	**N_{60}**	**$(N_1)_{60}$**	**Pocket Pen., tsf**	**Est. Equiv., Qu, tsf**	637.70	25.6-35.0'	Brown coarse sand, with scattered fine gravel, medium dense.
30.0	6-7-5	15	18	Sand		628.0	35.0-45.0'	Gray coarse sand, medium dense.
40.0	6-7-7	17	17	Sand		618.30	45.0-51.5'	Gray fine sand, dense.
50.0	12-16-22	46	40	Sand		611.80	51.5-52.0'	Cobbles and gravel.
53.5	14-28-42	85		4.70		611.30	52.0-57.5'	Gray and purple claystone, soft.
75.0	82 in 5"	100.00		9.0+	6.0	605.80	57.5-62.6'	Greenish-gray clay shale, soft, dark brown
95.0	82 in 3"	100.00		9.0+	9.7			claystone seam from 60.0 to 60.2'.
						600.70	62.6-63.2'	Dark gray claystone, soft.
						600.10	63.2-63.6'	Black bituminous coal, thickly laminated
								and brittle.
						599.70	63.6-67.0'	Dark gray, thinly laminated clay shale
								and claystone, soft to medium hard.
						596.30	67.0-67.3'	Black coal, brittle, soft.
						596.0	67.3-67.6'	Gray silt shale, moderately hard.
						595.70	67.6-74.8'	Light gray claystone, soft.
						588.50	74.8-75.4'	Coal, soft, brittle.
						587.90	75.4-77.3'	Gray clay shale, soft.
						586.0	77.3-82.9'	Black carbonaceous clay shale, medium hard.

CORING LOG (NX Double Tube Barrel)								
From	**To**	**Run**	**Rec**	**Loss**	**% RQD**			
55.0	60.0	5.0	5.0	0.00	0.00			
60.0	65.0	5.0	4.90	0.10	0**			
65.0	75.0	10.0	7.80	2.20	0.00			
75.40	80.40	5.0	5.0	0.00	0.00			
80.40	85.40	5.0	3.90	1.10	0.00			
WATER TABLE OBSERVATIONS								
Date	**Time Change**	**Depth Hole Open**		**Depth To Water**				
								**RQD on limestone portion only.

N_{60} = (Em/60)Nm N_{60} - Corrected N value for standard 60% SPT efficiency. Em - Measured transfer efficiency in percent. Nm - Observed N-value.

$(N_1)_{60}$ = Normalized standardized blow count corrected for effective overburden pressure.

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   and by judgment of the operator. THIS INFORMATION IS FOR DESIGN PURPOSES ONLY.

MISSOURI DEPARTMENT OF TRANSPORTATION
Division of Materials

BORING DATA (CORE & SPT)

Job No.	J2P0639				Design	A5910
County	Carroll/Lafayette		Route	65.00	Skew	Right Angles
Over	Missouri River				Skew	Right Angles
Logged by	Davis				Operator	Lamberson
Equipment	Failing 1500				Drillers Hole No.	L-00-14
Hole Stab. by	Casing				Date of Work	04/25/00
Automatic Hammer Efficiency			73.00	%	Drill No.	G-7889

Bent	Station	Location	Elevation		LOG OF MATERIALS *	
11.00	95+63	56' LT.				
Offset to:	95+60.8	56.6' LT.				
TEST DATA			580.40	82.9-86.6'	Black coal and carbonaceous shale	
Depth, ft. / **SPT Blows/6"** / **N60** / **(N1)60** / **Pocket Pen., tsf** / **Est. Equiv., Qu, tsf**					(probable loss from this interval), soft, brittle.	
			576.70	86.6-87.9'	Light gray claystone, soft (underclay).	
			575.40	87.9-99.3'	Dark gray, thinly laminated silt shale, micaceous, medium hard.	
			564.0	99.3-100.9'	Light gray limestone, thick bedded, coarse grained, medium hard.	
			562.40	100.9-105.0'	Brown and gray coarse grained sandstone, hard.	
			558.30	105.0-105.6'	Brownish-gray claystone, hard.	
			557.70		Boring terminated.	

TEST DATA

Depth, ft.	SPT Blows/6"	N_{60}	$(N_1)_{60}$	Pocket Pen., tsf	Est. Equiv., Qu, tsf

UNCONFINED COMPRESSIVE STRENGTH TEST DATA

Elev.	Depth, ft.	Qu, tsf	P.P., tsf
603.8	59.4	4.1	9.0+
600.8	62.4	5.6	9.0+
594.2	69.0	3.1	9.0+
592.2	71.0	4.0	9.0+
583.8	79.4	9.2	9.0+
582.2	81.0	38.1	9.0+
571.2	92.0	7.2	9.0+
573.4	89.8	2.1	9.0+
565.2	98.0	7.5	9.0+

CORING LOG (NX Double Tube Barrel)

From	To	Run	Rec	Loss	% RQD
85.40	90.40	5.0	3.9	1.10	
90.40	95.40	5.0	5.0	0.00	
95.60	100.60	5.0	5.0	0.00	
100.60	105.60	5.0	5.0	0.00	

WATER TABLE OBSERVATIONS

Date	Time Change	Depth Hole Open	Depth To Water

N_{60} = (Em/60)Nm N_{60} - Corrected N value for standard 60% SPT efficiency. Em - Measured transfer efficiency in percent. Nm - Observed N-value.

$(N_1)_{60}$ = Normalized standardized blow count corrected for effective overburden pressure.

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L-00-14
Bent 11 95+60.8 56.6' Lt.
4/25/00

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MISSOURI DEPARTMENT OF TRANSPORTATION
Division of Materials
BORING DATA (CORE & SPT)

Job No.	J2P0639							
County	Carroll/Lafayette			Route	65.00	Design	A5910	
Over	Missouri River					Skew	Right Angles	
Logged by	Hilchen/Stevens					Operator	Lamberson	
Equipment	Failing 1500					Drillers Hole No.	L-00-12	
Hole Stab. by	Casing					Date of Work	04/24/00, 04/25/00	
Automatic Hammer Efficiency					73.00 %	Drill No.	G-7889	

Bent	Station	Location	Elevation	LOG OF MATERIALS *	
11.00	95+70	42' LT.	663.20	Inaccessible due to difficulty in positioning barge.	
Offset to:	95+64	35.5' LT.	663.20	0.0-6.1'.	Barge deck.

		TEST DATA				657.20	6.1-24.0'	Water.
Depth, ft.	SPT Blows/6"	N_{60}	$(N_1)_{60}$	Pocket Pen., tsf	Est. Equiv., Qu, tsf	639.20	24.0-30.0'	Gray to tan fine to medium sand, with trace gravel.
35.0	8-8-7	18	19	Sand		633.20	30.0-38.0'	Gray fine sand, medium dense, trace black
45.0	4-6-7	16	15	Sand				lignite fines, fine gravel from 41.6 to 42.3'.
54.0	27-39-43 in 5.5"	100		9.0+	3.9	625.20	38.0-51.1'	Gray medium sand, medium dense.
75.50	82 in 5"	100.00		9.0+	6.0	612.10	51.1-58.6'	Gray and purple claystone, soft.
95.90	82 in 4"	100.00		9.0+	7.4	604.60	58.6-63.8'	Greenish-gray clay shale, soft.
						599.40	63.8-64.3'	Gray to dark gray clay shale, soft to medium hard.
						598.90	64.3-64.4'	Black coal.
						598.80	64.4-67.3'	Gray claystone, poorly laminated, soft to medium hard.
						595.90	67.3-68.1'	Black carbonaceous clay shale, medium hard.
						595.10	68.1-68.4'	Black coal, brittle.
						594.80	68.4-70.5'	Gray claystone, soft.
						592.70	70.5-78.3'	Gray claystone to clay shale, soft, coal at 74.5'.

		CORING LOG (NX Double Tube Barrel)				584.90	78.3-85.9'	Black carbonaceous shale, medium hard.
From	To	Run	Rec	Loss	% RQD	582.30	85.9-87.7'	Black coal, fossiliferous, brittle, medium hard.
55.50	60.50	5.0	5.0	0.00		575.50	87.7-99.9'	Black carbonaceous shale, medium hard.
60.50	65.50	5.0	4.80	0.20		563.60	99.9-101.8'	Brown and gray medium to coarse grained,
65.50	70.50	5.0	4.70	0.30				fossiliferous limestone, medium bedded, hard.
70.50	75.50	5.0	5.0	0.00		561.40	101.8-104.9'	Brown and gray coarse grained sandstone,
75.90	80.90	5.0	4.20	0.80				hard.
80.90	85.90	5.0	5.0	0.00		556.30	104.9-105.4'	Dark brown claystone, medium hard.
85.90	90.90	5.0	2.70	2.30		557.80		Boring terminated.
90.90	95.90	5.0	5.0	0.00				
96.30	105.40	9.1	9.10	0.00				

	WATER TABLE OBSERVATIONS					
Date	Time Change	Depth Hole Open	Depth To Water			

N_{60} = (Em/60)Nm N_{60} - Corrected N value for standard 60% SPT efficiency. Em - Measured transfer efficiency in percent. Nm - Observed N-value.

$(N_1)_{60}$ = Normalized standardized blow count corrected for effective overburden pressure.

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MISSOURI DEPARTMENT OF TRANSPORTATION
Division of Materials

BORING DATA (CORE & SPT)

Job No.	J2P0639					Design	A5910
County	Carroll/Lafayette		Route	65.00		Design	A5910
Over	Missouri River					Skew	Right Angles
Logged by	Hilchen/Stevens					Operator	Lamberson
Equipment	Failing 1500					Drillers Hole No.	L-00-12
Hole Stab. by	Casing					Date of Work	04/24/00, 04/25/00
Automatic Hammer Efficiency				73.00 %		Drill No.	G-7889

Bent	Station	Location	Elevation		LOG OF MATERIALS *		
11.00	95+70	42' LT.	663.20				
Offset to:	95+64	35.5' LT.	663.20				

TEST DATA							
Depth, ft.	SPT Blows/6"	N_{60}	$(N_1)_{60}$	Pocket Pen., tsf	Est. Equiv., Qu, tsf		

			UNCONFINED COMPRESSIVE STRENGTH TEST DATA			
			Elev.	Depth, ft.	Qu, tsf	P.P., tsf
			603.8	59.4	4.1	9.0+
			600.8	62.4	7.2	9.0+
			594.2	69.0	5.8	9.0+
			591.2	72.0	4.3	9.0+
			584.7	78.5	3.4	9.0+
			569.6	93.6	10.4	9.0+
			565.1	98.1	11.1	9.0+

CORING LOG (NX Double Tube Barrel)							
From	To	Run	Rec	Loss	% RQD		

WATER TABLE OBSERVATIONS					
Date	Time Change	Depth Hole Open	Depth To Water		

N_{60} = (Em/60)Nm N_{60} - Corrected N value for standard 60% SPT efficiency. Em - Measured transfer efficiency in percent. Nm - Observed N-value.

$(N_1)_{60}$ = Normalized standardized blow count corrected for effective overburden pressure.

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L-00-12
Bent 11 95+64 35.5' Lt.
4/24/00

MISSOURI DEPARTMENT OF TRANSPORTATION
Division of Materials

BORING DATA (CORE & SPT)

Job No.	J2P0639				**Design**	A5910
County	Carroll/Lafayette		**Route**	65.00	**Skew**	Right Angles
Over	Missouri River				**Operator**	Lamberson
Logged by	Davis				**Drillers Hole No.**	L-00-11
Equipment	Failing 1500				**Date of Work**	04/24/00
Hole Stab. by	Casing				**Drill No.**	G-7889
Automatic Hammer Efficiency				73.00 %		

Bent	Station	Location	Elevation	LOG OF MATERIALS *	
11.00	95+77	56' LT.			
Offset to:	95+78.5	55.3' LT.	663.20	0.0-6.1'	Barge deck.

TEST DATA

Depth, ft.	SPT Blows/6"	N_{60}	$(N_1)_{60}$	Pocket Pen., tsf	Est. Equiv., Qu, tsf	Elevation		
						657.10	6.1-28.6'	Water.
						634.60	28.6-51.3'	Gray coarse grained sand, dense to very dense.
30.0	40-41 in 4"	100	118	Sand	3.9	611.90	51.3-53.2'	Gray thinly to thickly laminated silt shale,
40.0	2-3-7	12	12					weathered, soft, cut with rockbit
50.0	14-20-21	50	44					from 51.5 to 53.2'.
63.20	39-43 in 4"	100		9.0+	4.0	610.0	53.2-58.5'	Gray and purple poorly laminated
84.0	82 in 4"	100		9.0+	7.4			claystone, medium hard.
						604.70	58.5-61.7'	Greenish-gray thinly laminated clay shale,
								soft, brown claystone seam from 59.6 to 60.0'
						601.50	61.7-66.7'	Gray poorly laminated claystone,
								soft to medium hard.
						596.50	66.7-68.2'	Black bituminous coal, medium hard, brittle.
						595.0	68.2-69.8'	Gray micaceous silt shale, medium hard.
						593.40	69.8-76.2'	Gray poorly laminated claystone,
								soft to medium hard.

CORING LOG (NX Double Tube Barrel)

From	To	Run	Rec	Loss	% RQD	Elevation		
						587.0	76.2-81.6'	Dark gray to black clay shale, medium hard
53.20	58.20	5.0	0.00	0.00				to moderately hard.
58.20	63.20	5.0	0.00	0.00		581.60	81.6-85.5'	Black, brittle, thinly bedded to thickly
64.0	74.0	10.0	8.40	1.60				laminated bituminous coal.
74.0	79.0	5.0	5.0	0.00		577.70	85.5-90.4'	Gray thinly to thickly laminated calcareous
79.0	84.0	5.0	4.10	0.90				sandy silt shale, medium hard, brittle.
84.30	89.30	5.0	4.50	0.50		572.80	90.4-99.5'	Gray thinly laminated silt shale,
89.30	94.30	5.0	5.0	0.00				soft to medium hard.
94.30	99.30	5.0	5.0	0.00		563.70	99.5-100.8'	Gray medium bedded coarse grained
99.30	104.30	5.0	5.0	0.00	100**	562.40	100.8-104.3'	limestone, moderately hard to hard. Brown and gray coarse grained sandstone,

WATER TABLE OBSERVATIONS

Date	Time Change	Depth Hole Open	Depth To Water	Elevation		
				558.90		medium to moderately hard. Boring terminated.
						**RQD on limestone portion only.

N_{60} = (Em/60)Nm N_{60} - Corrected N value for standard 60% SPT efficiency. Em - Measured transfer efficiency in percent. Nm - Observed N-value.

$(N_1)_{60}$ = Normalized standardized blow count corrected for effective overburden pressure.

* Persons using this information are cautioned that the materials shown are determined by the equipment noted and accuracy of the "log of materials" is limited thereby and by judgment of the operator. THIS INFORMATION IS FOR DESIGN PURPOSES ONLY.

MISSOURI DEPARTMENT OF TRANSPORTATION
Division of Materials

BORING DATA (CORE & SPT)

Job No.	J2P0639					
County	Carroll/Lafayette		**Route**	65.00	**Design**	A5910
Over	Missouri River				**Skew**	Right Angles
Logged by	Davis				**Operator**	Lamberson
Equipment	Failing 1500				**Drillers Hole No.**	L-00-11
Hole Stab. by	Casing				**Date of Work**	04/24/00
Automatic Hammer Efficiency				73.00 %	**Drill No.**	G-7889

Bent	Station	Location	Elevation	LOG OF MATERIALS *
11.00	95+77	56' LT.		
Offset to:	95+78.5	55.3' LT.		

TEST DATA

Depth, ft.	SPT Blows/6"	N_{60}	$(N_1)_{60}$	Pocket Pen., tsf	Est. Equiv., Qu, tsf	

UNCONFINED COMPRESSIVE STRENGTH TEST DATA

Elev.	Depth, ft.	Qu, tsf	P.P., tsf
607.2	56.0	4.1	9.0+
601.8	61.4	4.7	9.0+
598.2	65.0	2.1	9.0+
592.8	70.4	10.2	9.0+
587.2	76.0	2.9	9.0+
570.2	93.0	9.1	9.0+
561.7	101.5	56.7	9.0+

CORING LOG (NX Double Tube Barrel)

From	To	Run	Rec	Loss	% RQD

SIEVE ANALYSIS (PERCENT PASSING)

AASHTO T88

	Depth, ft.		
	30	40	50
3/4"	100	100	100
3/8"	100	99	99
No. 4	99	99	98
No. 10	98	98	97
No. 16	96	94	96
No. 40	68	16	88
No. 50	46	8	82
No. 100	12	3	57
No. 200	9	2	20

WATER TABLE OBSERVATIONS

Date	Time Change	Depth Hole Open	Depth To Water

N_{60} = (Em/60)Nm N_{60} - Corrected N value for standard 60% SPT efficiency. Em - Measured transfer efficiency in percent.   Nm - Observed N-value.

$(N_1)_{60}$ = Normalized standardized blow count corrected for effective overburden pressure.

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362

L-00-11
Bent 11 95+78.5 55.3' Lt.
4/24/00

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MISSOURI DEPARTMENT OF TRANSPORTATION
Division of Materials

BORING DATA (CORE & SPT)

Job No.	J2P0639				**Design**	A5910	
County	Carroll/Lafayette		**Route**	65.00	**Skew**	Right Angles	
Over	Missouri River				**Operator**	Wineland	
Logged by	Stevens				**Drillers Hole No.**	L-00-10	
Equipment	Failing 1500				**Date of Work**	04/24/00	
Hole Stab. by	Casing				**Drill No.**	G-7889	
Automatic Hammer Efficiency				73.00 %			

Bent	Station	Location	Elevation		LOG OF MATERIALS *
11.00	95+77	28' LT.	663.30	0.0-6.1'	Barge deck.
			657.20	6.1-25.7'	Water.

TEST DATA

Depth, ft.	SPT Blows/6"	N_{60}	$(N_1)_{60}$	Pocket Pen., tsf	Est. Equiv., Qu, tsf	Elevation		LOG OF MATERIALS *
						637.60	25.7-47.0'	Gray medium to coarse grained sand,
								with light brown to gray silt, dense.
30.0	31-16-9	30	35			616.30	47.0-51.2'	Gray fine grained clayey sand, dense.
40.0	9-9-11	22	22			612.20	51.2-56.9'	Gray and purple claystone, soft.
50.0	4-5-17	27	24			606.40	56.9-62.7'	Greenish-gray clay shale, soft,
72.70	62-20 in 1.5"	100		9.00+	4.3			claystone seam at 60.4'.
88.40	82 in 4"	100		9.00+	7.4	600.60	62.7-67.0'	Gray to dark gray clay shale,
								soft to medium hard.
						596.30	67.0-67.7'	Black carbonaceous clay shale,
								medium hard.
						595.60	67.7-70.1'	Gray claystone, poorly laminated, soft.
						593.20	70.1-70.8'	Coal.
						592.50	70.8-73.8'	Gray silt shale to siltstone, medium hard.
						589.50	73.8-74.4'	Coal.
						588.90	74.4-78.2'	Gray clay shale, medium hard.
						585.10	78.2-85.2'	Black carbonaceous clay shale,

CORING LOG (NX Double Tube Barrel)

From	To	Run	Rec	Loss	% RQD	Elevation		LOG OF MATERIALS *
								medium hard.
52.70	57.70	5.0	5.0	0.00		578.0	85.2-91.4'	Black coal and carbonaceous shale,
								soft, brittle.
57.70	62.70	5.0	4.50	0.50		571.80	91.4-99.3'	Black carbonaceous clay shale,
62.70	67.70	5.0	5.0	0.00				moderately hard.
67.70	72.70	5.0	4.60	0.40		564.0	99.3-100.5'	Brown and gray, medium to coarse grained,
73.40	78.40	5.0	5.0	0.00				irregularly bedded, fossiliferous limestone,
78.40	83.40	5.0	5.0	0.00				hard, fair quality.

WATER TABLE OBSERVATIONS

Date	Time Change	Depth Hole Open	Depth To Water	Elevation		LOG OF MATERIALS *
				560.80	100.5-103.8'	Gray and brown coarse grained sandstone,
						thin bedded, hard, well to moderate
						cementation.
				559.50		Boring terminated.

N_{60} = (Em/60)Nm N_{60} - Corrected N value for standard 60% SPT efficiency. Em - Measured transfer efficiency in percent.   Nm - Observed N-value.

$(N_1)_{60}$ = Normalized standardized blow count corrected for effective overburden pressure.

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   and by judgment of the operator. THIS INFORMATION IS FOR DESIGN PURPOSES ONLY.

MISSOURI DEPARTMENT OF TRANSPORTATION
**Division of Materials**
BORING DATA (CORE & SPT)

Job No.	J2P0639						
County	Carroll/Lafayette		Route	65.00	Design	A5910	
Over	Missouri River				Skew	Right Angles	
Logged by	Stevens				Operator	Wineland	
Equipment	Failing 1500				Drillers Hole No.	L-00-10	
Hole Stab. by	Casing				Date of Work	04/24/00	
Automatic Hammer Efficiency				73.00 %	Drill No.	G-7889	

Bent	Station	Location	Elevation	LOG OF MATERIALS *			
11.00	95+77	28' LT.					

TEST DATA							
Depth, ft.	SPT Blows/6"	N_{60}	$(N_1)_{60}$	Pocket Pen., tsf	Est. Equiv., Qu, tsf		
						UNCONFINED COMPRESSIVE STRENGTH TEST DATA			
						Elev.	**Depth, ft.**	**Qu, tsf**	**P.P., tsf**
						607.4	55.9	0.8	6.5
						602.8	60.5	3.6	9.0+
						599.1	64.2	3.1	9.0+
						585.9	77.4	3.3	9.0+
						582.9	80.4	13.9	9.0+
						568.4	94.9	8.1	9.0+

CORING LOG (NX Double Tube Barrel)							
From	To	Run	Rec	Loss	% RQD		
83.40	88.40	5.0	3.60	1.40			
88.8	98.80	10.0	10.0	0.00			
98.8	103.8	5.0	4.50	0.50	72.00		

WATER TABLE OBSERVATIONS							
Date	Time Change	Depth Hole Open	Depth To Water				

$N_{60} = (Em/60)Nm$ N_{60} - Corrected N value for standard 60% SPT efficiency. Em - Measured transfer efficiency in percent. Nm - Observed N-value.

$(N_1)_{60}$ = Normalized standardized blow count corrected for effective overburden pressure.

* Persons using this information are cautioned that the materials shown are determined by the equipment noted and accuracy of the "log of materials" is limited thereby and by judgment of the operator. THIS INFORMATION IS FOR DESIGN PURPOSES ONLY.

365

L-00-10
Bent 11 95+77 28' Lt.
4/24/00

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APPENDIX K
GEOLOGY FOR SOUTH ABUTMENT AT LEXINGTON SITE

The Cherokee Group is overlain by the Marmaton Group. The Marmaton Group contains more limestone units than the Cherokee Group and was encountered in the subsurface investigation for the piers located on the bluff near the south abutment. The Marmaton Group is comprised of two subgroups, the Fort Scott Subgroup and the Appanoose Subgroup. The Fort Scott Subgroup includes four formations from the base upwards: the Excello, Blackjack Creek, Little Osage, and Higginsville: formations as described below.

Excello Formation: The Excello Shale consists of dark gray shale with green shale partings, (Thompson 1995). This formation was encountered from about elevation 201 to 201.9 m and averages about 0.8 m (2.6 ft) in thickness.

Blackjack Creek Formation: The Blackjack Creek Limestone consists of a lower and an upper unit of earthy limestone (Thompson 1995). This formation was encountered from about elevation 201.9 to 203 m and averages about 1.1 m (3.6 ft) in thickness.

Little Osage Formation: The Little Osage Formation includes a thinly laminated calcareous shale, poorly laminated clay shale, (probably underclay), the Summit Coal Bed, a black carbonaceous shale, and a gray thick bedded limestone (Houx Limestone Member), (Thompson 1995). The Houx Limestone member was encountered at about elevation 206 m. The Houx Limestone was overlain by gray to dark gray or reddish brown shale. The upper part of the Little Osage Formation was light gray and tan shale with light brown laminations. This formation was encountered from about elevation 203 to 212.1 m and averages about 9.1 m (29.9 ft) in thickness.

Higginsville Formation: The Higginsville Limestone consists of a light gray, fine grained, thin to medium bedded limestone. This formation was encountered from about elevation 212.1 to 213.6 m and averages about 1.5 m (4.9 ft) in thickness.

The subsurface investigation for the piers on the bluff south of the river encountered four of the seven widely-recognized successions of the Appanose Subgroup of the Marmaton Group. These are from the base upward: Labette Formation, Pawnee Formation, Bandera Formation, and the Altamont Formation. The Altamont Formation is overlain by about 5.5 m (18.0 ft) of eolian loess, a wind blow soil. The Labette Formation was encountered during excavation of pier 25 and the Bandera and Altamont Formations were encountered at pier 26.

Labette Formation: The Labette Formation consists of an underclay, the Alvis Coal bed, a dark gray fossiliferous shaly limestone, and the Lexington Coal Bed (Thompson 1995). The Lexington Coal Bed was mined from the 1860's to the 1920's and the end abutment (bent 26) is located directly over Riverton Mine No. 2. This formation was encountered from about elevation 213.6 to 215.8 m and averages about 2.2 m (7.2 ft) in thickness.

Pawnee Formation: The Pawnee Formation consists of a dark gray to black fissile shale (Anna Shale Member), a gray thin bedded limestone (Myrick Station Limestone Member), dark gray shale (Mine Creek Shale Member), and a medium to thick bedded Limestone (Coal City Limestone Member), (Thompson 1995). The Mulberry Coal Bed was not encountered in the borings. This formation was encountered from about elevation 215.8 to 220.8 m and averages about 5.0 m (16.4 ft) in thickness.

Bandera Formation: The Bandera Formation consists of a gray to brown shale overlain by purple shale. The purple shale is overlain by gray to brown shale to sandstone (Bandera Quarry Member), (Thompson 1995). This formation was encountered from about elevation 220.8 to 226.1 m and averages about 5.3 m (17.3 ft) in thickness.

Altamont Formation: The Altamont Formation consists of three members from the base upwards: the Amoret Limestone Member, the Lake Neosho Shale Member, and the Worland Limestone Member (Thompson 1995). Only the Amoret Limestone and the Lake Neosho Shale were encountered in the borings from about elevation 227.4 to 226.1 m. The formation averages about 1.3 m (4.3 ft) in thickness.

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